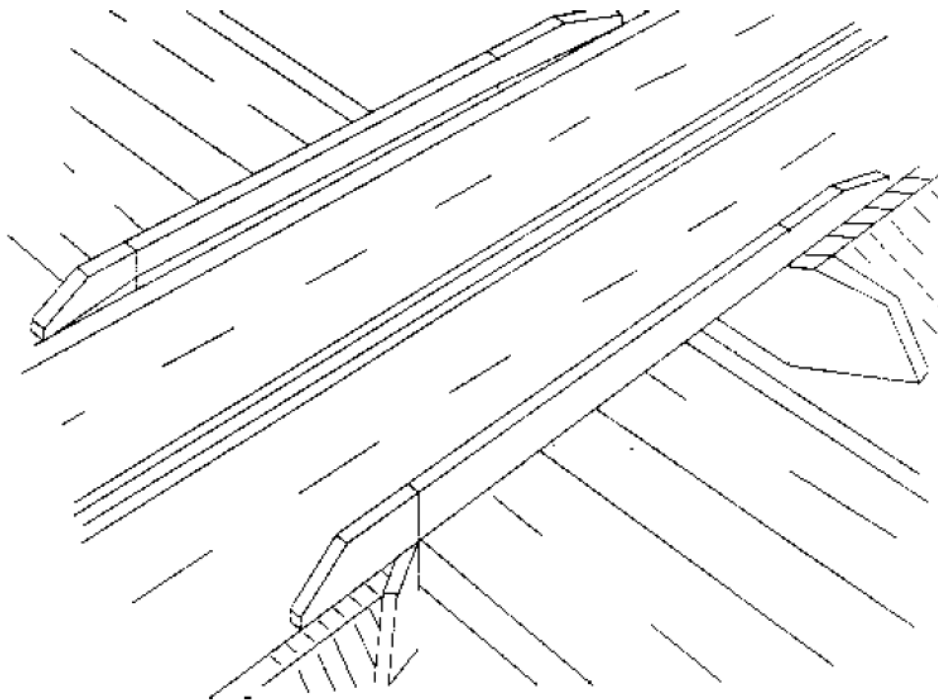


DESIGN MANUAL PART 4



STRUCTURES

PROCEDURES – DESIGN – PLANS PRESENTATION
PDT – PUB No. 15M

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

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PART 4

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DESIGN MANUAL

PART 4

INTRODUCTION

Design Manual, Part 4 is part of a series of Department (Pennsylvania Department of Transportation) design manuals which have the specific objective of obtaining uniformity and establishing standard policies and procedures in the preparation of design and construction plans for highway structures. The provisions of the AASHTO LRFD Bridge Design Specifications, shall govern where applicable, except as specifically modified by the requirements herein.

The inclusion of specified design criteria in this Manual does not imply that existing roadways, which were designed and constructed using different criteria, are either substandard or must be reconstructed to meet the criteria contained herein. Many existing facilities which met the design criteria at the time of their construction are adequate to safely and efficiently accommodate current criteria unless a safety or capacity problem manifests itself.

Since it is not feasible to provide a highway system that is continuously in total compliance with the most current design criteria, it is imperative that both new construction and reconstruction projects are selected from a carefully planned program which identifies those locations in need of improvement and then treats them in priority order. Once a new construction or reconstruction project is selected in this manner, this Manual should be used as a guide in determining the appropriate design criteria to be used.

The Design Manual has precedence over AASHTO design specifications and bridge design and construction standards.

The Design Manual, which is published in loose-leaf form to facilitate changes and expansion, is divided into three parts: (A) "Policies and Procedures", (B) "Design Specifications" and (C) "Appendices to Design Manual, Part 4".

In Part B, the AASHTO LRFD article numbering system is followed. Where a new article has been added, the suffix P, to designate "Pennsylvania Article", appears at the end of the new article number.

All references to AASHTO LRFD Bridge Design Specifications sections, articles, equations, figures or tables carry the prefix A. References to AASHTO commentary carry the prefix AC. References to Design Manual, Part 4, Part A, "Policies and Procedures", carry the prefix PP. References to Design Manual, Part 4, Part B, "Design Specifications", carry the prefix D. References to commentary to Design Manual, Part 4 carry the prefix DC.

When a DM-4 article modifies and/or adds information to an AASHTO article, the first sentence of the DM-4 article shall read "The following shall supplement Ax.x.x". When a DM-4 article replaces an AASHTO article, the first sentence shall read "The following shall replace Ax.x.x".

Policy and procedure information which does not have a corresponding AASHTO article number will be placed using a "0" attached to the article number.

Maintenance and updating of this Manual is the responsibility of the Bureau of Project Delivery.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

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LIST OF ABBREVIATIONS

AAR	Association of American Railroads
AASHTO	American Association of State Highway and Transportation Officials - Generally "AASHTO" is a reference to <u>LRFD Bridge Design Specifications</u>
ADT	Average Daily Traffic
ADTT	Average Daily Truck Traffic
AITC	American Institute of Timber Construction
AREMA	American Railway Engineering and Maintenance of Way Association
ASTM	American Society of Testing and Materials
AWS	American Welding Society
BC-Stds.	Bridge Construction Standards
BD-Stds.	Bridge Design Standards
BBSS	Bureau of Business Solutions and Services
BPAA	Bridge Plan Approval Application
BMS	Bridge Management System
BOPD	Bureau of Project Delivery
BDTD	Bridge Design and Technology Division
BRADD	Bridge Automated Design and Drafting
Bul. 15	Approved Construction Materials (Bulletin 15)
CADD	Computer-Aided Design and Drafting
C/C	Center-to-Center
CVN	Charpy V-notch (Impact Requirement)
DCNR	Department of Conservation and Natural Resources
DEP	Department of Environmental Protection
DGCE	District Grade Crossing Engineer
DPE	District Project Engineer
DM-4	Design Manual, Part 4
DPM	District Project Manager
FAST	Facilitate Acceleration through Special Techniques

LIST OF ABBREVIATIONS (continued)

FHWA	Federal Highway Administration
HDD	Highway Delivery Division
LFTF	Liquid Fuel Tax Funded
LRFD	Load and Resistance Factor Design
LTS	Laboratory Testing Section
MPS	Master Policy Statement
MSE	Mechanically Stabilized Embankment
NBIS	National Bridge Inspection Standards
PC	Personal Computer
PennDOT	Pennsylvania Department of Transportation
PMC	Project Management Committee
P/S	Prestressed
PS&E	Plans, Specifications and Estimate
PTF	Pennsylvania Traffic Factor
PUC	Public Utility Commission
Q/A	Quality Assurance
RC	Reinforced Concrete
RC-Std.	Roadway Construction Standards
RQD	Rock Quality Designation
SCEF	Structural Committee for Economical Fabrication
SIRS	Structure Inventory Record System
SMS	Structural Materials Section
S. R.	State Route
STStd.	Structural Standards (Predecessor to BC Series)
TC-8700 Series	Traffic Control Signing Standards
TS&L	Type, Size and Location
VEACTT	Value Engineering Accelerated Construction Technology Transfer

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INCORPORATED STRIKE-OFF LETTERS

SOL 431-98-23

Highway Hydraulics
Hydraulic Engineering Circular No. 23
FHWA HI 97-030

SOL 431-09-14

Publication 15M, Design Manual Part 4
Change No. 1

SOL 431-10-6

Fill and Cover Requirements for Pipe Revisions to Design and Construction Publications

SOL 431-10-12

Bridge Design Standards, BD-600M Series (Pub. 218M): September 2010 Edition

SOL 431-10-13

Bridge Construction Standards, BC-700M Series (Pub. 219M): October 2010 Edition

SOL 431-11-4

Publication 218M BD-642M, BD-643M, BD-644M, BD-645M, & BD-647M and Publication 15M Design Manual, Part-4 –
Revised Definition of Design Sign Area and Design Preference for Overhead Sign Structures

SOL 431-11-6

Modifications to Publication 15M, Design Manual Part 4
Implementation Measures to Mitigate Corrosion of Substructure J-Bar Reinforcement

SOL 431-11-8

Publication 15M, Design Manual Part 4
Addition of Section B 2.9P and Quality Assurance Form D-519 to Design Manual Part 4 for Construction Loading on Bridges

SOL 431-11-9

Publication 15M, Design Manual Part 4
Revision of Appendix J - Approved Commercially Available or Consultant-Developed Software

SOL 431-12-2

Implementation of AAAP Cement Concrete

SOL 431-12-3

Publication 219M, BC-701M - Protective Fence -
New Protective Fence Details for Bridges over Railroads

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1

PART A: POLICIES AND PROCEDURES

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
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PART 4

VOLUME 1

PART A: POLICIES AND PROCEDURES

PREFACE

All references to AASHTO LRFD Bridge Design Specifications sections, articles, equations, figures or tables carry the prefix A.

References to AASHTO commentary carry the prefix AC.

References to Design Manual, Part 4, Part A, "Policies and Procedures", carry the prefix PP.

References to Design Manual, Part 4, Part B, "Design Specifications", carry the prefix D.

References to commentary to Design Manual, Part 4 carry the prefix DC.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
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PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

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1.1 APPLICABLE SPECIFICATIONS AND STANDARDS

1.1.1 Design Specifications

The following specifications, unless otherwise modified or amended in this manual, shall govern the design of highway structures:

- (1) AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010
- (2) AASHTO/AWS D1.5M/D1.5:20XX - Bridge Welding Code {XX per Publication 408, Section 1105.03(m)}
- (3) AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, 4th Edition 2001 including Interims through 2006.
- (4) AREMA Manual for Railway Engineering, 2011, and other specific requirements by the operating railroad
- (5) AITC, American Institute of Timber Construction, Timber Construction Manual, Third Edition, 1985
- (6) AASHTO Guide Specifications for the Design of Stress - Laminated Wood Decks, April 1991
- (7) AASHTO Guide Specification for Strength Design of Truss Bridges (Load Factor Design), 1985.

1.1.1.1 AASHTO Interim Specifications

As AASHTO Interim Specifications are published, BDTD will review the interims and incorporate them into this manual as necessary.

1.1.1.2 Deviations From Specifications

Any deviations from the specifications and standards listed above, or the Department's design criteria described hereafter, require the Chief Bridge Engineer's approval. The approved design criteria shall be shown on the bridge plans.

1.1.1.3 Order Of Precedence

The design criteria given in this Manual supersedes any criteria given in the referenced design specifications in PP1.1.1 and PP1.1.3.

In case of conflict or where clear precedence cannot be established, the Chief Bridge Engineer shall establish governing specifications.

1.1.1.4 Interpretation Of Design Specifications

For Design Manual, Part 4, and AASHTO LRFD Bridge Design Specifications, the final interpretation shall be made by the Chief Bridge Engineer.

1.1.1.5 Applicable Specifications for Local Projects

Local projects may be designed using AASHTO specifications only, unless the local authority agrees to use the Department's specifications in addition to the AASHTO specifications. Additionally, Publication 70M, Guidelines for the Design of Local Roads and Streets, may be used.

1.1.1.6 AASHTO Guide Specifications**1. Guide Specifications for Fatigue Design of Steel Bridges, 1989**

This specification shall not be used.

The current Design Manual, Part 4, in combination with the AASHTO LRFD Bridge Design Specifications provides all needed criteria and directions.

2. Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges, 1989

This specification shall not be used.

The current PennDOT Bridge Inspection Manual in combination with the AASHTO Manual for Bridge Evaluation provides adequate criteria and directions.

The Department may ask any designer or bridge strength evaluator to use any of these publications under a special situation. The Chief Bridge Engineer should be contacted prior to authorization of their usage.

3. Guide Specifications for Bridge Railing, 1989

This specification shall not be used.

Only use Department established standard details for bridge railing and transition. Any needed modification will be published as a modification to the Department's railing standards. Special designs must match the Department's railing shape and size and ultimate strength.

4. Guide Specifications for Design and Construction of Segmental Concrete Bridges, 1999, including the 2003 Interim

This specification, with the exception of the following items, is to be used in preparation of segmental concrete projects:

- a. Corrosion protection of prestressing steel shall be provided in accordance with Publication 408, Section 1108.03(b).
- b. Provision for deck replacement and full-depth repair method must be presented. Provide a provision for a 1 1/4 in. latex overlay.
- c. Do not use unbonded prestressing system as a permanent prestressing system.
- d. Prior to starting design of any component of a segmental bridge, applicable section(s) of these specifications and design methodology must be reviewed and approved by the Chief Bridge Engineer.

5. Guide Specifications for Structural Design of Sound Barriers, 1989 including the 1992 and 2002 Interims

Refer to PP3.6.4 for further guidelines.

6. Guide Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections, 1991

This specification shall not be implemented and is not to be used as a design guide on Department projects.

7. Guide Specifications for Aluminum Highway Bridges, 1991

The specification may be used for information only.

Availability of the specification is not an endorsement to use aluminum bridges on Department projects. Special approval of the Chief Bridge Engineer is required for aluminum usage in bridges, as well as specific parts of the specifications on a project-by-project basis.

A copy of the Guide Specifications for Aluminum Highway Bridges is available in the BDTD.

8. Guide Specifications for Fatigue Evaluation of Existing Steel Bridges, 1990

This specification shall not be used on Department projects.

9. LRFD Guide Specifications for the Design of Pedestrian Bridges, 2009

This specification shall be used for bridges intended for only pedestrian, equestrian, light maintenance vehicles, and/or bicycle traffic.

10. Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011

1.1.1.7 AASHTO Standard Specifications

1. Standard Specifications for Highway Bridges, 17th Edition

This specification shall be used when a service load or load factor design are requested, i.e. rehabilitation projects.

2. Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, 4th Edition, 2001

Implement the interims through 2006 for this specification.

1.1.2 Construction Specifications

1. Publication 408, Specifications

2. Changes to Publication 408

1.1.3 Bridge Standards

The following standard drawings shall be followed and used wherever practicable:

1. Standards for Bridge Construction, BC Series

2. Standard Plans for Low Cost Bridges, BLC Series (Only use on local bridge projects and designs. Details must be upgraded to comply with the current policy.)

3. Standards for Bridge Design, BD Series

4. Standards for Roadway Construction, RC Series

5. Traffic Control and Signing Standards, TC-8600 and TC-8700 Series

6. Lighting, TC-8715

Reference to any or all of the Standard Drawings in the BC Series, on the structure design drawings, in lieu of showing specific details is encouraged, provided coordinating information is shown on the design drawings. Reference to applicable RC Standards shall also be made.

1.1.3.1 Supplemental Drawings

The referenced Standard Drawings shall be listed in the Table for Supplemental Drawings and in the General Notes on the Roadway Plans with the other referenced drawings.

Lists of past and current bridge standards are given in Appendix C for information.

1.2 MODIFICATIONS TO DESIGN MANUAL

Whenever a user believes that modifications (including additions) to Design Manual, Part 4, would improve the present design practice, the following sections outline the course of action to be taken.

1.2.1 Submission of Recommended Modifications

The recommended modification shall be transmitted to the Director, Bureau of Project Delivery, by completing the revision request form shown in Figure 1.2.1-1.

	DESIGN MANUAL, PART 4	TO:
	REVISION REQUEST	_____, DIRECTOR BUREAU OF PROJECT DELIVERY
REQUESTING ORGANIZATION/COMPANY	DATE OF REQUEST	SIGNATURE OF PERSON REQUESTING CHANGE
NAME OF PERSON TO CONTACT	TELEPHONE NUMBER	ADDRESS
IF YOU REQUIRE MORE SPACE THAN IS PROVIDED, USE SHEETS OF PLAIN PAPER THAT ARE THE SAME SIZE AS THESE SHEETS, NUMBER YOUR ANSWERS TO CORRESPOND WITH THE WAY THE ITEMS ARE NUMBERED BELOW.		
1. TITLE, SECTION, NUMBER AND PAGE NUMBER OF THE EXISTING SPECIFICATION.		
2. RECOMMENDED MODIFICATION AND THE CHAPTER(S) OR SECTION(S) INTO WHICH IT SHOULD BE INCORPORATED.		
3. REASON AND/OR EXPLANATION FOR THE MODIFICATION.		

Figure 1.2.1-1 – Revision Request Form

Figure 1.2.1-1 – Revision Request Form (continued)

1.2.2 Processing of Recommended Modifications

Upon receiving the proposed modification, the Director, Bureau of Project Delivery, will take the following action:

- (a) The Director, Bureau of Project Delivery, will review the recommended modification and transmit copies to the various Bureau Directors and District Bridge Engineers involved for their comments.
- (b) Upon receiving comments, the Director, Bureau of Project Delivery, will finalize the modification and take appropriate action, including obtaining comments by means of a Clearance Transmittal letter and securing FHWA approval if applicable.
- (c) If modifications are accepted, the revised or added page(s) will be assigned a revision date which will be noted on the upper right-hand corner of the page. Revised pages will be distributed periodically.
- (d) If the proposed modification is not accepted, the Director, Bureau of Project Delivery, will notify the originator of the reasons for rejecting it.

1.3 BRIDGE DESIGNS AND REVIEWS BY CONSULTANTS

The Quality Assurance Form for Engineering Agreements, in accordance with Appendix A, must be completed and included as a part of the technical proposal for an Engineering Agreement or as a part of Work Order under an Open-End Contract. This form shall be incorporated in an appendix along with the District's Scope of Work. A separate form must be completed for each structure included under the technical proposal or Work Order.

1.3.1 Scope of Work for New Designs

The scope of work for bridge projects shall contain the following bridge-related items as applicable:

- (a) Hydraulic and hydrologic report, if applicable, and acquisition of all needed waterway-related permits, including permits for temporary stream crossing.
- (b) Type, size and location studies, cost estimates, foundation exploration and recommendations, final design, and special provision preparation.
- (c) Shop drawing review.
- (d) Review of alternate design developed by contractors and Design-Build projects.
- (e) Consultation during construction.
- (f) Constructibility review by an independent consultant for all new major, unusual and complex bridges.

Constructibility is defined as the optimum use of construction knowledge and experience in planning, design, procurement and field operations to achieve overall project objectives and construction without any delay, structural integrity problems, or major claims.

The constructibility review should begin as early as the conceptual design stage (i.e., TS&L stage) to minimize redesign costs, project delays or the inclusion of undesirable details in the contract documents.

1.3.2 Scope of Work for Rehabilitation Projects

The following items shall be included as applicable:

- (a) Deck condition survey, or bridge inspection and recommendations if the needed detailed information is not available from the regular NBIS report
- (b) Petrographics for concrete

- (c) Borings through piers and/or abutments to determine (1) bottom of footings, (2) material type and quality, (3) foundation materials, (4) foundation type, etc., if substructure units are to be reused
- (d) Tests for steel to establish chemical composition, yield and ultimate strengths, as well as Charpy tests
- (e) Fatigue damage analysis with possible strain gaging if decision on rehabilitation vs. replacement is involved
- (f) Rating analysis
- (g) Constructibility review by an independent consultant for all new major, unusual and complex bridge rehabilitation projects.

1.3.3 Selection of Design Methodology

The Department shall specify the design method (line girder method, two-dimensional grid analysis, or three-dimensional finite element design) to be used for the project.

For straight girder bridges with skews greater than 70°, a line girder method (with distribution factors as given in A4.6.2.2 and D4.6.2.2) shall be used (see PP3.2.2 for the Department's definition of skew angle).

1.3.4 Bridge Design Review by Consultants

The consultants may be asked to perform one of the following two different levels of review:

- (a) Level 1 review (see PP1.3.4.1 for additional details) is applicable mostly to contractor-designed alternates and design-build projects of a complex nature or to those which incorporate "leading edge" technology. Such designs will often be beyond the scope of the criteria covered by the Design Manual and the Department's standards. The review consultants will be required to make a detailed review of the design to ensure that the Contractor's conceptual design approved by the Department is correctly developed and presented by the Contractor. This level of review will also be applicable to "leading edge" designs prepared by other consultants retained by the Department, where an independent review by another consultant would be in the best interest of the Department.

Figure 1.3.4-1, Figure 1.3.4-2 or Figure 1.3.4-3 shall be used for stamping the first sheet of the bridge plans reviewed by the review consultants.

- (b) Level 2 review (see PP1.3.4.1 for additional details) is applicable to routine types of structures designed in conformance with Department criteria and standards. Review shall be limited to ensuring general conformance with the Department's design criteria and standards.

Figure 1.3.4-4 or Figure 1.3.4-5 shall be used for stamping the first sheet of the bridge plans reviewed by the review consultants.

The depth of review required of the review consultant shall be described in detail in the Engineering Agreement. When the level of review cannot be clearly determined for contractor-designed alternates and design-build projects, one of the two levels shall be assumed on the basis of the complexity of the as-designed bridge. The level of review will be changed, depending upon the alternate design proposed by the low bidder, either by the District Bridge Engineer or the Chief Bridge Engineer, according to approval responsibility. (Note that if a contractor-designed alternate or a design-build project converts a routine type of structure to a "leading edge" type, the Chief Bridge Engineer becomes responsible for approval.) Since most of the review work assigned to consultants is based upon a specific rate of pay, only the total cost of review would change, depending upon the level of review. When the level of review cannot be readily determined, the cost for both levels may be sought during the agreement stage.

Review may be assigned to consultants when work loads are such that review cannot be done in-house. Most consultant review assignments will be made for the review of contractor-designed alternates and design-build projects, but there may also be some assignments for the review of designs prepared by other consultants during the design phase.

Alternate Design S x x x x A Reviewed by: PE Seal Review Consultant's Name, Signature and Date The design review is a detailed review for compliance with the contract documents and for proper development and presentation of the Contractor's conceptual design approved by the Department. It is not intended to relieve the Contractor of full responsibility for accuracy and completeness of the plans or for complete compliance with the contract documents.

Figure 1.3.4-1 – Level 1 Review - Applicable to Complex and "Leading-Edge" Contractor-Designed Alternates and Design-Build Projects

Design reviewed by: PE Seal Review Consultant's Name, Signature and Date The design review is a detailed review for proper development and presentation of the concepts in the type, size and location plans approved by the Department. It is not intended to relieve the designer of full responsibility for the proper development and presentation of the design and for the accuracy and completeness of the plans.

Figure 1.3.4-2 – Level 1 Review - Applicable to Complex and "Leading-Edge" Designs Prepared by Other Consultants Retained by the Department

Design reviewed by: PE Seal Review Consultant's Name, Signature and Date The design review is a detailed review for compliance with the contract documents, the Department's design and construction criteria and standards, and for proper development and presentation of the concepts in the type, size and location plans approved by the Department. It is not intended to relieve the Contractor of full responsibility for the proper development and presentation of the design, the accuracy and completeness of the plans, or for complete compliance with the contract documents.

Figure 1.3.4-3 – Level 1 Review - Applicable to Design-Build projects where the TS&L plan has been provided

Alternate Design	
S x x x A	
Reviewed by:	PE Seal
Review Consultant's Name, Signature and Date	
The design review is for general conformance with the contract documents and the Department's design and construction criteria and standards. It is not intended to relieve the Contractor of full responsibility for the accuracy and completeness of the plans or for complete compliance with the contract documents.	

Figure 1.3.4-4 – Level 2 Review - Applicable to Non-complex Contractor-Designed Alternates and Design-Build Projects Designed in Accordance with the Department's Criteria and Standards

Design reviewed by:	
PE Seal	
Review Consultant's Name, Signature and Date	
The design review is for general conformance with the Department's design and construction criteria and standards and is not intended to relieve the designer of full responsibility for the accuracy and completeness of the plans.	

Figure 1.3.4-5 – Level 2 Review - Applicable to Non-complex Structures Designed by Other Consultants in Accordance with the Department's Criteria and Standards

1.3.4.1 Review Levels

- (a) Level 1 - Detailed review shall consist of the following, as applicable, and additional requirements which may be unique to a particular bridge:
 - (1) Evaluation of design methods and design assumptions.
 - (2) Evaluation of computer program used in design (or check of design using a different computer program acceptable to the Department).
 - (3) Check of manual calculations.
 - (4) Check of construction methods, including applicable safety regulations, when required, to ensure that the intent of the design can be realized.
 - (5) Check of erection stresses, where applicable.
 - (6) Check of plans to ensure that design information is adequately and correctly presented.
 - (7) Check of construction dimensions is not required, except as in (6) above unless specified in the engineering agreement.
 - (8) Quantity check is not required.
 - (9) Constructibility check is not required for contractor-designed alternates and Design-Build projects, except as noted in (4) above.
 - (10) Constructibility check is required for review of design prepared by the consultants retained by the Department.
 - (11) Review for cost-effectiveness when design is prepared by another consultant retained by the Department.
 - (12) Review for compliance with Department criteria and standards, as applicable.
- (b) Level 2- Review for conformance to Department criteria and standards shall consist of the following, as applicable, and additional requirements which may be unique to a particular bridge:
 - (1) Review for compliance with Department criteria and standards.
 - (2) Review for constructibility and cost-effectiveness when design is prepared by a consultant retained by the Department.
 - (3) Check of design calculations only when required (will depend on quality of design, history of design consultant, etc.).
 - (4) Check of plans to ensure that design information is adequately and correctly shown.
 - (5) Check of construction dimensions and quantities are not required unless specified in the engineering agreement.

1.4 COMPUTER PROGRAMS

1.4.1 Modification, Acquisition, or Development of Programs

The software outlined in PP1.4.2 and PP1.4.7 shall be used for all PennDOT projects. If software for a particular application is not available from the Department, the designer may use other commercially available software with the approval of the Department. The designer is fully responsible for the entire design and analysis, regardless of the software used.

An Engineering District may request modification to the existing bridge engineering programs or acquisition or development of new engineering programs from the Director, Bureau of Project Delivery. Proper justification, benefits, etc. shall be incorporated in the request.

The Bureau of Project Delivery in coordination with the Bureau of Business Solutions and Services will have the software developed, acquired or modified, conduct acceptance testing and provide or arrange for training, if needed.

1.4.2 Bridge Automated Design and Drafting System (BRADD)

The BRADD software was written as a tool for the LRFD design of simple span concrete, steel and P/S concrete bridges with or without sidewalks with span lengths ranging from 18 ft. to 200 ft.. The software supports tangent geometry, horizontal curves (chord beams) and vertical curves, with a bridge skew range from 25 degrees to 90 degrees. Available cross-section types are normal, symmetrical, superelevation, and superelevation transition, with a maximum of 8 design lanes or 20 beams in the cross section for a maximum width of 125.54 ft..

1.4.2.1 Department Use

- (a) Each Engineering District shall be responsible for designing bridges using BRADD on its CADD domain. The District Bridge Engineer shall be responsible for keeping a log of the projects for which BRADD is used and the savings realized by using BRADD.
- (b) Any programming or technical errors found during use of the software shall be brought, in writing, to the attention of the Chief Bridge Engineer, Bureau of Project Delivery. Appropriate changes will be made by the BRADD Manager, Bureau of Project Delivery, with the concurrence of the Chief Bridge Engineer. The Chief Bridge Engineer will notify all the Districts of corrections.
- (c) Since BRADD can provide plans for more than one bridge type, the Districts are encouraged to develop designs for several of the most economical types of bridges. Cost analysis for different bridge types may be run on BRADD to select bridge types prior to generating the drafting files. This type of run shall be made if the designer is unsure of the most economical bridge types. Plans for more than one bridge type shall be provided whenever feasible in the bid document package to increase competition, thus reducing construction cost.

1.4.2.2 Use of BRADD on Consultant Designed State Projects

The Department requires the use of BRADD on all single span bridge projects, including all single span PennDOT Bridge Projects (new single span bridge projects and all single span bridge superstructure replacement projects) unless justified by the designer.

There is a high degree of interest in utilizing BRADD for developing bridge plans for one span bridge replacement projects. However, BRADD can also be partially used for multiple span bridges since it can design and draft abutments for such bridges.

The District Bridge Engineer shall be responsible for keeping a record of the projects for which BRADD is used and the savings realized by using BRADD.

The BRADD Software, based on the Load and Resistance Factor Design method is available only in a Windows based (Windows XP Professional Edition with Service Pack 3, Windows Vista, and Windows 7) version on CD.

The Department will provide this software to consultants and any outside agencies for a fee, provided they sign a license agreement for its use. The Pennsylvania Department of Transportation is the only acquiring agent for this software. Anyone interested in obtaining a copy of the license agreement and fee schedule should contact the BRADD Manager, Bureau of Project Delivery at (717) 787-7057 or view information on the Department website at www.dot.state.pa.us.

The department will provide e-mail notification, e-notification, to individuals who register at <http://BRADD.ENGPROGRAMS.com>. A company or organization can register as many individuals as desired. E-notifications will deliver timely non-policy issues in a relatively informal manner, such as: Problem reports, work-around solutions, upcoming version releases, answers to frequently asked questions and issues deemed current and relevant to proper implementation of BRADD.

1.4.2.3 Use of BRADD on Non-Departmental Pennsylvania Bridge Projects

The Department requires the use of BRADD on all non-Departmental Pennsylvania single span bridge projects funded in part by the Department unless justified by the designer. The Department encourages the use of BRADD on all other single span non-

PennDOT Pennsylvania Bridge Projects (new single span projects and all single span bridge superstructure replacement projects).

There is a high degree of interest in utilizing BRADD for developing bridge plans for one-span bridge replacement projects. However, BRADD can also be partially used for multiple span bridges since it can design and draft abutments for such bridges.

The District Bridge Engineer shall be responsible for keeping a record of the projects reviewed and approved for which BRADD is used and the savings realized using BRADD.

The BRADD Software, based on the Load and Resistance Factor Design method is available only in a Windows based (Windows XP Professional Edition with Service Pack 3, Windows Vista, and Windows 7) version on CD.

The Department will provide this system to consultants and any outside agencies for a fee, provided they sign a license agreement for its use. The Pennsylvania Department of Transportation is the only acquiring agent for this software. Anyone interested in obtaining a copy of the license agreement and fee schedule should contact the BRADD Manager, Bureau of Project Delivery at (717) 787-7057 or view information on the Department website at www.dot.state.pa.us.

The department will provide e-mail notification, e-notification, to individuals who register at <http://BRADD.ENGPROGRAMS.com>. A company or organization can register as many individuals as desired. E-notifications will deliver timely non-policy issues in a relatively informal manner, such as: Problem reports, work-around solutions, upcoming version releases, answers to frequently asked questions and issues deemed current and relevant to proper implementation of BRADD.

1.4.3 CADD Cells for Bridge Details

The Engineering District may develop a standard bridge detail library (cells) for its own use. However, any detail that has statewide application shall be sent to the Director, Bureau of Project Delivery, for approval and permanent storage in the statewide cell library.

1.4.4 Bridge Engineering Software on PCs

If a District desires to develop or secure bridge engineering software (executables) for PC use, the software must be tested and accepted by the Bureau of Project Delivery prior to its use in bridge engineering.

1.4.5 Commercial Software

Commercially available or consultant-developed software shall be submitted to the Bureau of Project Delivery for review upon request. The submission shall include, as a minimum, design and/or analysis methodology, assumptions, capabilities, limitations, special instructions and comparison with Department-reviewed software. The Department will maintain and provide, upon request, a list of reviewed programs. Additional information for review, including software access if requested, shall be provided to ensure proper evaluation by the Department.

Appendix J contains a list of acceptable girder analysis programs for LRFD design.

The Department has the discretion to either accept or reject the use of any commercially available or consultant-developed software proposed for use on any project. The Bureau of Project Delivery is available to help the Districts in this matter by providing guidance and recommendations when requested. In any and all cases, the design consultant is responsible for the accuracy of any and all computer software programs utilized on a project.

1.4.6 Computer Programs for Girder Bridges

Any computer program which has not been reviewed by the Department shall be submitted to, and approved by, the Chief Bridge Engineer prior to its use. A sample bridge(s) selected by the Department is to be modeled with the program so that the Department can make comparisons between its reviewed programs and the proposed program. The submission shall contain a description of how the program models the bridge, a discussion of any unique or special features of the program, input data sheets, output data sheets, a discussion of how the live load is treated, and a summary of the results, which should include (but not be limited to) the following:

- (a) Dead load longitudinal moment diagram
- (b) Positive and negative live load, plus impact longitudinal moment envelope
- (c) Dead load shear diagram

- (d) Positive and negative live load impact shear envelope
- (e) Table of reactions
- (f) Table of bottom flange lateral moments which occur at the maximum positive and negative longitudinal moments in each span of a curved bridge
- (g) Table of diaphragm loads for curved and/or skewed bridges

This data shall not have load factors applied. The responsibility for obtaining approval of a computer program falls upon the person who has submitted it to the Bureau of Project Delivery.

1.4.7 PC Versions of the PennDOT Engineering Programs

The following PC versions of the PennDOT engineering programs, based on the general principle of engineering, will be available for use:

<u>Program Name</u>	<u>Program Title</u>
BSP	Beam Section Properties (dual units)
CAMBR	Field Check of Camber (dual units)
CBA	Continuous Beam Analysis (dual units)
CLLMR	Comparison of Live Load Moments and Reactions (dual units)
EngAsst	Engineering Assistant
GRPRO	Grade Profile (dual units)
HGEO	Horizontal Geometry
BRGEO	Bridge Geometry (dual units)

The following PC versions of the PennDOT engineering programs, based on the Load and Resistance Factor Design method and working in U. S. Customary units, will be available for use:

<u>Program Name</u>	<u>Program Title</u>
ABLRFD	LRFD Abutment and Retaining Wall Analysis and Design
BPLRFD	LRFD Bearing Pad Design and Analysis
BXLRFD	LRFD Box Culvert Design and Rating
FBLRFD	LRFD Floorbeam Analysis and Rating
PAPIER	Pennsylvania Pier Analysis
PSLRFD	LRFD Prestressed Concrete Girder Design and Rating
SPLRFD	LRFD Steel Girder Splice Design and Analysis
STLRFD	LRFD Steel Girder Design and Rating
TRLRFD	LRFD Truss Analysis and Rating (not available yet)

The following PC versions of the PennDOT engineering programs, based on the Service Load and/or Load Factor Design method and work only in the Customary U. S. units, are available for use:

<u>Program Name</u>	<u>Program Title</u>
ABUT5	Abutment and Retaining Wall
ARCH	Arch Analysis and Design
BAR7	Bridge Analysis and Rating
BOX5	Box Culvert Design and Rating
PS3	Prestressed Concrete Girder Design and Rating
SIGN	Sign Structure Analysis

All programs are only available in a PC version running Microsoft Windows (Windows XP, Windows Vista, and Windows 7) operating system. They are available on CD.

The Pennsylvania Department of Transportation is the only acquiring agent for these programs. The Department will provide these programs to consultants, local government (such as county and township) and educational institution for a fee per program, provided they sign a license agreement for the use of these programs. The Department will also provide these programs to federal or state agencies for free, provided they sign the license agreement for the use of these programs. Consultants, local government, and educational institution interested in obtaining a copy of the license agreement and program fees should view the information on the PennDOT Engineering Programs official website at <http://penndot.engrprograms.com/home/>. The forms for Software Update Request and Request for PennDOT's Engineering Software License can be downloaded from above web site.

The Department will provide e-mail notification, e-notification, to individuals who register at <http://Penndot.engrprograms.com>. A company or organization can register as many individuals as desired. E-notifications will deliver timely non-policy issues in a relatively informal manner, such as; computer program incident reports, work-around solution, upcoming program version deliveries, answers to frequently asked questions and issues deemed current and relevant to proper implementation of the Department's Engineering Software.

1.5 MAJOR, UNUSUAL OR COMPLEX BRIDGES

1.5.1 Definitions

Major bridges – Major bridges are defined as bridges estimated to cost \$15 million or more. This criterion also applies to individual units of separated dual bridges.

Unusual bridges – An unusual bridge is one with difficult or unusual foundation problems, new or complex designs involving unusual structures or operational features, or bridges for which the design standards or criteria may not be applicable. Use of new products and experimental or demonstration projects are also considered as unusual structures.

Complex bridges – Complex bridges are stayed girder bridges, segmental bridges, any structure having a clear unsupported length in excess of 500 ft., or bridges classified as complex by the Chief Bridge Engineer on the basis of TS&L or conceptual review.

1.5.2 Design Requirements

For 100% State funded projects, with the exception of interstate bridges, one complete design shall be developed for a bridge, using either steel or concrete or a combination of both, whichever is determined to be most economical at the type, size and location studies step.

The designer of major, unusual (including new products, experimental, or demonstration features), or complex structures, regardless of funding, shall include inspection and maintenance instructions as a part of the original design. Critical details and special inspection and maintenance requirements shall be stipulated on the plans. The plans shall also include details of access for inspection of the subject details.

For major, unusual (including new products, experimental, or demonstration features), or complex bridges, indicate on the plans or in special provisions the complete erection plan, detailed erection procedures with all needed survey controls, and all required computations.

1.5.3 Early Involvement of BDTD

If the District desires, the Bureau of Project Delivery, Bridge Design and Technology Division will provide its assistance in the consultant selection process for major, complex or unusual structures. For this purpose, a major bridge is defined as a bridge having an estimated construction cost of \$15 million or more. The BDTD will assist in reviewing and commenting on the scope of work up to and including the final prioritization of the short-listed firms by the District.

Early involvement will be beneficial as the BDTD will gain familiarity with the project.

This assistance is voluntary.

1.6 PLAN PRESENTATION

The following shall generally apply to all Department projects. For non-Federally funded municipal projects, the governing municipality may have different requirements. Municipal requirements for Federally funded projects shall comply with stewardship and oversight agreement.

1.6.1 Drawings

Design drawings for highway structures shall be prepared on original reproducible drawings furnished by the Department, or on an approved equivalent Standard ANSI D size 34" x 22" with a border approximately 1 ½ in. on the left and 1/2 in. on the other three sides. A space shall be reserved at the bottom-right corner of each sheet for the title block. (For dimensions and other details see Table 1.6.1-1 and Figure 1.6.1-1.)

Table 1.6.1-1 – Bridge Drawings Block Dimensions

DESCRIPTION	WIDTH	HEIGHT
Title Block (Standards for Bridge Construction, Standards for Bridge Design)	8"	4 3/4"
Professional Engineer's Seal Block (and Prepared By)	6"	3"
Professional Engineer's Seal Block (block only)	3"*	3"*
Bridge Supplemental Drawing Block (1/4@ line spacing)	8"	OPEN
Bridge Revision Block	7 1/2"	OPEN
*For BRADD, general plan sheet block will be 2 3/8" x 3". For all other BRADD sheets, the sheet block will be 2 3/8" x 2 3/8".		

USE BLACK OPAQUE INK

3"

1/4"

6"

8"

7 1/2"

4 3/4"

8"

REVISIONS

Mark	Description	By	Chk'd	Rec'd	Date
REVISIONS					

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
CLARION COUNTY
S.R. 80 SEC. 8
SEG. 410 OFFSET 0 N.B. SEG. 411 OFFSET 11 S.B.
S.R. 80-8 STA. 58+60 N.B. & 59+50 S.B.
OVER B & O R.R. & BEAVER CREEK
DUAL 3-SPAN STEEL I-BM. BRIDGES
FRAMING PLAN

RECOMMENDED _____ SHEET 1 OF 8
BRIDGE ENGINEER _____ S- 7460

DESCRIPTION DWG. NO. APP. DATE
SUPPLEMENTAL DRAWINGS

PA. P.E. SEAL
PREPARED BY
XYZ ENGINEERING COMPANY
123 AB STREET
CITY, STATE
SIGNATURE & DATE

Figure 1.6.1-1 – Example of Dimensions and Details for Design Drawings

1.6.2 Title Blocks

Refer to Figure 1.6.2-1.

Mark	Description	By	Chk' d.	Recm' d.	Date
REVISIONS					

COMMONWEALTH OF PENNSYLVANIA DEPARTMENT OF TRANSPORTATION	
CLARION COUNTY S.R. 80 SECTION 8 SEG. 410 OFFSET 0 N.B. SEG. 411 OFFSET 11 S.B. S.R. 80-8 STA. 1+786.128 N.B. & 1+813.560 S.B. OVER B & O R.R. & BEAVER CREEK DUAL 3-SPAN STEEL I-BM. BRIDGES FRAMING PLAN	
RECOMMENDED _____ ** BRIDGE ENGINEER	SHEET <u>1</u> OF <u>8</u> * S-7460

* Show the following on the first sheet only:

+ SUPPLEMENTAL DRWGS.

** Show BRIDGE ENGINEER or CHIEF BRIDGE ENGINEER on the first sheet only.

Figure 1.6.2-1 – Example of Title Block

1.6.2.1 Basic Information

The outline of the title block, dividing lines, and information basic to all projects will be reprinted on the sheets furnished by the Department or CADD generated sheets meeting the requirements of PP1.6.1. The other necessary information shall be added by the designer and shall be shown on each sheet of design drawings as applicable.

The county, route, section, segment, offset, station, features intersected, sheet number and S-number shall be shown in a manner similar to the example in Figure PP1.6.2-1.

The offset is to the beginning of the bridge, and the station shown should be at the approximate center of the bridge or at intersecting base lines as applicable. Segments and offsets for existing bridges shall be consistent with the BMS database.

1.6.2.2 Description of Structure

The number of spans and a brief description of the bridge type (steel, prestressed concrete, etc.) shall be shown below the general information in the title block.

1.6.2.3 Description of Drawings

A brief description of what is contained on the sheet should be shown below the description of the structure for each sheet of drawings, e.g., General Plan, General Notes and Quantities, Abutment No. 1, Pier No. 3, Framing Plan Spans 10 and 11, etc.

1.6.2.4 Sheet Numbering

Each sheet of a set of drawings shall be numbered consecutively beginning with No. 1. Sheets shall be arranged in a logical (title sheet, general plan, stake-out, substructure, superstructure, etc.) and orderly manner beginning with a general plan or an index sheet and ending with boring logs. Sheets closely related to one another shall be grouped together. Sheets that are added as a revision during construction shall bear the same number as the most closely related sheet, with an appropriate suffix (R1, R2, R3, etc.) added.

1.6.2.5 Structure Plan Number (S-Number)

An automated S-numbering System, S-NUM (S-Number Update Method), is to be used by District personnel to provide a computerized method of securing and recording structure plan numbers (S-numbers).

The District is responsible to access the system and obtain the S-number. To access S-NUM, a TSO User ID Account is required and a system manual is available from BDTD.

BDTD does have the capability to modify S-NUM permanent records, should any misrepresentation of information occur, or if the District changes its mind on a project, or its time schedule, etc.

All structure plan numbers, whether they are designed by Consultant or District, will receive an S-number in the District Office.

Plans do not need an S-number until the Final Plan Stage, permitting time for the District to enter data. However, it is preferable to secure a number at the TS&L approval stage.

For Design-Build projects, the S-number shall be established for the Conceptual TS&L during preliminary design. Districts should contact the BDTD to update the S-NUM permanent records with any final structure information during final design.

Generally, there should be a separate set of drawings for each structure, except for dual structures and in special cases where it is more convenient to include more than one structure on a set of drawings. Rehabilitation or repair drawings, even if they are only sketches made on or 8 ½" x 11" (U.S. Customary Letter Size) sheets, shall have S-numbers assigned to them. When a structure has two designs in different materials for the purpose of encouraging competition, each design shall have a complete set of drawings and the same S-number, except for suffixes A and B, respectively, for each set. For a contractor-designed alternate, the S-number will be the same as the original design, except with suffix A added if there was no suffix on the original drawings, and suffix C added if there were two designs with suffixes A and B on the respective original drawings. The S-number will become part of the BMS database through regular input channels.

Structure Drawings with an S-number are required for all culverts having a clear span of 8 ft. or more. If a multiple pipe, box, or arch culvert exists, the span length is the combined clear span opening.

Regardless of funding source, for local jurisdiction bridges, the municipality shall submit the type, size, location and foundation proposal, along with one copy of Form D-4274 (Application for Bridge Plan Approval) to the District Executive. The District Bridge Engineer will assign a structure number to this form in accordance with Publication 9, Chapter 3.

Sign structure plans require an S-number but do not need to be signed by the District Bridge Engineer.

VMS/DMS sign structures require S-numbers and are to be signed by the Chief Bridge Engineer. For Title Blocks for sign structures and VMS/DMS sign structures, refer to Design Manual, Part 3, Chapter 8.

Sound barrier and proprietary wall plans require an S-number. When contractor-designated alternate plans are submitted for proprietary walls, the S-number, shown on the original design of the proprietary wall, shall be suffixed by the letter P and should be shown on the alternate design plans.

Structure-mounted sound barrier shall be considered an integral part of the structure. Therefore, it will have the same S-number.

1.6.3 Special Requirements

1.6.3.1 Engineering Seal

For drawings prepared by a consultant, the structure drawings shall be prepared by, or under the direct supervision of, a professional engineer registered in Pennsylvania. The consultant's name and address together with the professional engineer's seal of the responsible professional engineer in charge of the design, his or her signature, and the date shall be shown in black opaque ink near the bottom-middle of the first sheet of each set of structure drawings for the final submission of the plans to the Department.

For drawings prepared in-house, the structure drawings shall be prepared by, or under the direct supervision of, a professional engineer registered in Pennsylvania. The responsible Assistant District Bridge Engineer professional engineer's seal along with his or her signature and date shall be shown in black opaque ink near the bottom-middle of the first sheet of each set of structure drawings.

All other drawings in the set shall have the professional engineer's seal, and it shall be located near the title block. This seal may be either from a black ink rubber stamp seal or a facsimile seal.

The responsible professional geologist or professional engineer shall sign and seal the core boring drawings.

1.6.3.2 Supplemental Drawings

A table for supplemental drawings (standard drawings) shall be placed on the first sheet of each set of structure drawings immediately to the left of the title block whenever any standard drawings are referred to or are otherwise applicable to the construction of the structure. For BRADD the supplemental drawings block is placed in the top right corner of the Quantities sheet. The supplemental drawings shall be identified by drawing number, title and most recent recommendation date. (For an example, see Figure PP1.6.1-1.)

1.6.3.3 Revisions

A space shall be reserved immediately above the title block on all sheets of structure drawings for the purpose of listing revisions that are made to the drawings during construction. Drawings shall be laid out so as not to encroach on this area. Changes made to drawings prior to advertisement for letting shall not be listed as revisions. Changes made to drawings after a project is awarded for construction, including changes that are made by flyers prior to letting, shall be made by crossing out information that is being voided and adding the new information nearby, marked with a revision symbol and number. A brief description of the revision shall be listed in the revision block, together with the initials of those making the revision and checking it, and the date. The District Bridge Engineer has the responsibility of seeing that revisions are properly made.

1.6.3.4 Release of Information and Documents to Non-Department Source

The decision to release Department information in response to request from outside the Department is to be made by the respective Bureau Director, District Executive who has control of the material. It is Department policy that BMS tapes, bridge inspection reports, RMS files and accident reports are not released. Other information such as Preliminary Design Reviews, QA Reports, Material Vendor files, Contractor Pre-Qualification Files, and material related to claims are typically not released.

The Department may release bridge inventory information (such as location, size, type, etc.) only as determined by the Office of Chief Council. The Districts are encouraged to allow public inspection by prospective bidders of specific information for a given construction project during the timeframe from advertisement by a Notice to Contractors to bid opening for the project.

This inspection should consist of the ability of the contractor to view, but not copy, information used by the Department to make their engineering decisions which will assist the prospective bidder in making an informed bid. This may consist of the information submitted for foundations in accordance with PP1.9.4.3, PP1.9.4.4 and PP1.9.4.5, and information submitted for the TS&L regarding constructability issues and/or alternatives analyses.

The Department may also release engineering design computations to the successful bidder to assist in developing alternate design or value engineering redesign.

1.6.4 General Requirements

1.6.4.1 Layout

In the preparation of design drawings, every effort shall be made to draw the plans, sections, elevations and details accurately to scale. The scales shall be large enough to show clearly all dimensions and details necessary for construction of the structure. Preferably, plans, sections and elevations should be drawn to a scale not less than $1/4" = 1'-0"$ and details to a scale not less than $3/8" = 1'-0"$. Drawings shall be laid out in such a way that all details fall within the prescribed border lines. All detail views shall be placed on the drawing so as to allow adequate space between them and shall be drawn large enough to be easily read when reduced photographically by 50%. Refer to Design Manual, Part 3, for requirements to provide bar scales. Ensure slope designations are appropriate to U.S. Customary units, 3H:12V.

1.6.4.2 Linework and Lettering

All lines on the drawings shall be of sufficient density and width so as to have some residual density when reduced photographically by 50% and when microfilmed. Minimum height size for lettering, symbols and characters shall be $1/8$ in.. All characters shall be open, bold, uniform and formed with a dense, but not wide line. Space between the letters shall be one-half the width of the widest letter, and space between the lines of lettering shall be one-half the height of the tallest letter. For CADD-generated drawings, see Design Manual, Part 3, Chapter 12.

1.6.4.3 Checking

Each sheet of design drawings shall be thoroughly checked and initialed by the designer and the checker before being submitted for Department review. The designer and checker must be two separate individuals to maintain proper quality control of information shown on each sheet.

1.6.4.4 North Arrow

A north arrow symbol shall be placed on the General Plan and on all plan views of superstructure, substructure, or entire structure layout.

1.6.4.5 Dimensioning

Designers shall be particularly careful that sufficient overall and tie-in dimensions and geometric data are given on the plan. Tie-in and overall dimensions shall be arranged in such a way that it will not be necessary when reading the plans to add or deduct dimensions in order to determine the length, width, or height of any element of a structure. If "variable" dimensions are used, maximum and minimum values shall be provided.

1.6.4.6 Duplication of Details

Showing of details or dimensions in more than one place shall be kept to a minimum. Such duplication is usually unnecessary and increases the risk of errors, particularly when revisions are made. However, such details shall be appropriately cross-referenced so that they may be easily found.

1.6.4.7 Cross-Referencing

If, because of lack of space on a particular sheet or for convenience or other reasons, it is necessary to place a view or a section on another sheet or sheets, all such sheets shall be clearly cross-referenced by adding the appropriate sheet number after the section or detail designation, e.g., Section A-A (Sh __); Detail B (Sh __). Notes shall be used only if absolutely necessary.

1.6.4.8 Pay Limits

When misinterpretation is possible, the limits of pay items shall be clearly indicated on the corresponding details of a structure.

1.6.4.9 Abbreviations

Abbreviations of words shall generally be avoided, and those abbreviations which are not in common use shall be explained in a legend.

1.6.4.10 Reinforcement Bar Schedules

Bar marks should not be repeated. For bar marks that cover varying lengths of bar, the minimum and maximum lengths of bar shall be denoted in the schedules, along with the varying distance per number of bars. For example, S601, 9'-0" to 12'-0", vary 2 EA. by 6".

1.6.4.11 General Plan Sheet

The following essential information shall be shown on the first sheet, which is designated as "General Plan", unless a title sheet is used. If all of the following items cannot be accommodated on the first sheet, they may be shown on the next or succeeding sheets with proper reference.

(a) Plan

Outlines of substructure above ground and superstructure; length of spans along profile grade of roadway, skew angle(s), stations and grade elevations at intersections of profile grade with centerline bearing at abutment and centerlines at piers; stations at end and beginning of structure (see Figure 1.6.4.11-1), piers, abutments and wingwall designation (e.g., Pier 5, Abutment 1, Wingwall A); horizontal distance between profile grade lines in the case of dual structures; contours for existing and final groundlines; location of points of minimum actual and required vertical clearances, scuppers, and lighting poles; minimum actual and required horizontal clearances between underpassing highways or railroad tracks and faces of adjacent parts of substructure, and normal horizontal clearances between faces of substructure for drainage structures.

(b) Elevation

Rate and direction of roadway grade, spacing of railing posts, spacing and mounting heights of lighting poles, protective fence location, finished groundline and approximate original groundline along construction centerline of bridge, bottom of footing elevations, estimated pile tip elevations, and required and provided minimum vertical clearances together with the elevations that define the clearances provided. Show the type of joint and a movement classification for each joint on the plans. Show the bearing fixity at each substructure unit.

For definition and requirements for highway vertical clearance, see D2.3.3.2. For drainage structures, the minimum vertical clearance is the maximum unobstructed design flood flow depth under a bridge.

(c) Typical normal section(s) of superstructure

Roadway width between curbs or sidewalks, overall dimensions, out-to-out of outside faces of barriers, water tables, cross slopes of roadway, minimum slab thickness, girder spacing, girder type, girder size and overhang. All applicable cross-sections shall be shown on the general plan.

(d) Grade data

Horizontal and vertical alignment data, superelevation, run-in/run-out data, points of rotation, and provisions for spirals in accordance with Design Manual, Part 2.

(e) Deck elevations

On structures in which lanes converge or diverge, on structures with transition in superelevation or small skew angles (less than 75 degrees), or on curves or complicated layout, one or all of the following shall be shown at the direction of the District:

- Table showing elevations at gutter and breaks in cross slope of the deck at 10 ft. intervals
- Contour lines with framing plan superimposed
- Finished deck grade over the centerline of each girder at every tenth point starting from near centerline of the bearing of each girder to the far centerline of bearing.

(f) Summary of quantities

Pay item numbers and units shall be in accordance with the Department Specifications, Publication 408, or as specified and described in the Special Provision. For a list of common pay items, see the Construction Items Catalog (Publication 7) and the ECMS master items catalog. When two or more structures (such as dual bridges) are presented on the same plan, the quantities shall be tabulated for each structure. For presentation format see Figure 1.6.4.11-2. When a lump sum item is used, an approximate quantity shall be shown for the item.

All "AND Items", such as reinforcement, piles and pile tip reinforcements items, need not be repeated as shown in Figure 1.6.4.11-2 for BRADD projects. However, all "AND Items", except reinforcement bars and epoxy-coated reinforcement bars shall be repeated on the summary of quantities sheet (Plate B-II in Chapter 15 of Design Manual, Part 3) for each Contractor designed alternate option with the quantities to be inserted by the Contractor.

Design-build projects shall have pay items in accordance with PP1.11.2 and the Standard Special Provisions. The bid item for design is an individual pay item. The bid item for construction will be an "EITHER OR" format to distinguish material types. Refer to Figure 1.6.4.11-3 for sample tabulations.

(g) Index of sheets

Bridge sheet titles and numbers shall be listed when five or more drawing sheets are used.

(h) General notes (For a list of general notes, see PP1.7.)

(i) Hydraulic data (see PP7.1.2.)

For detailed submission items for each bridge submission, see PP1.9.

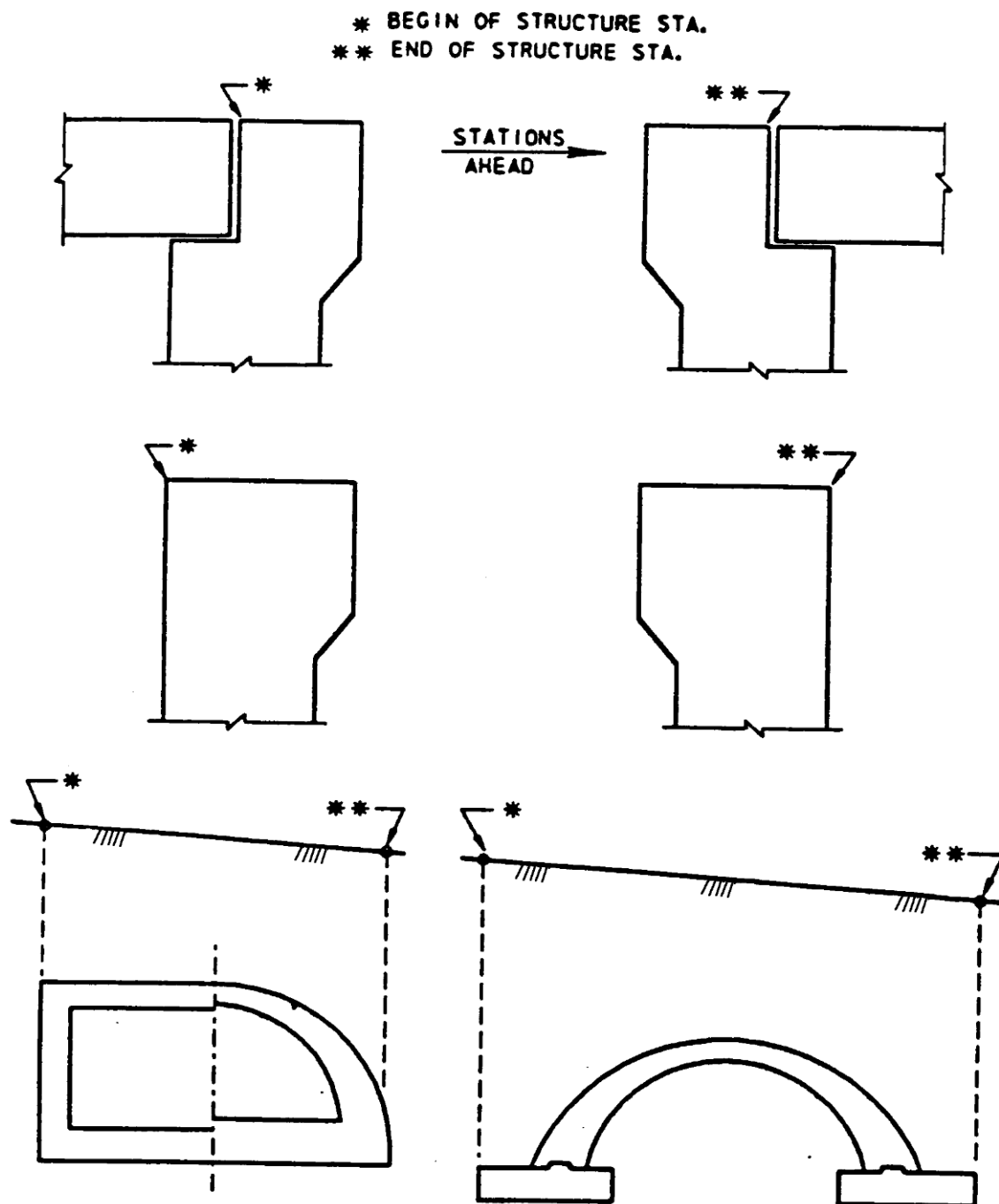


Figure 1.6.4.11-1 – Sections at End and Beginning of Structure

ALTERNATE STRUCTURE ITEMS			
ITEM NO.	DESCRIPTION	UNIT	TOTAL
8030-0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX	LS	LUMP SUM
8000-0001	PRESTRESSED CONCRETE BRIDGE STRUCTURE	LS	LUMP SUM
8100-0001	STEEL BRIDGE STRUCTURE	LS	LUMP SUM

APPROXIMATE QUANTITIES - BRIDGE STRUCTURE, AS DESIGNED						
ITEM NO.	DESCRIPTION	UNIT	ABUT. 1	ABUT. 2	SUPERSTRUCTURE	TOTAL
8030-0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX	LS				LS
(1)	CLASS 3 EXCAVATION	CY	0	7	---	7
(1)	CLASS AAAP CEMENT CONCRETE	CY	---	---	161 (2)	161
(1)	CLASS AA CEMENT CONCRETE	CY	22 (3)	4 (3)	44 (4)	70 (5)
(1)	CLASS A CEMENT CONCRETE	CY	179	103	---	282
(1)	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	CY	0	604	---	604
(1)	NO. 57 COARSE AGGREGATE	CY	3	3	---	6
(1)	PRESTRESSED CONCRETE BRIDGE BEAMS 28" x 73"	LF	---	---	921	921
(1)	NEOPRENE STRIP SEAL DAM, (3" MOVEMENT)	LF	35	---	---	35
(1)	STEEL BEAM TEST PILES, HP 12 x 53 (6)	---	LS (7)	LS (7)	---	---
(1)	GEOTEXTILE, CLASS 2, TYPE B	SY	87	132	---	219
(1)	6" STRUCTURE FOUNDATION DRAIN	LF	70	135	---	205
(1)	PROTECTIVE COATING FOR REINFORCED CONCRETE SURFACES (PENETRATING SEALERS, REINFORCED CONCRETE SUBSTRUCTURE SURFACES)	SY	122	123	---	245
AND						
1002-0001 (9)	REINFORCEMENT BARS	LB	16,574	8,701	0	25,275
AND						
1002-0053 (9)	REINFORCEMENT BARS, EPOXY-COATED	LB	1,879	454	22,666 (8)	24,999

Figure 1.6.4.11-2 – Tabulation of Bridge Bid Items and Approximate Quantities

APPROXIMATE QUANTITIES - BRIDGE STRUCTURE, AS DESIGNED						
ITEM NO.	DESCRIPTION	UNIT	ABUT. 1	ABUT. 2	SUPERSTRUCTURE	TOTAL
AND						
1002-0001 (9)	REINFORCEMENT BARS	LB	16,574	8,701	0	25,275
AND						
1002-0053 (9)	REINFORCEMENT BARS, EPOXY-COATED	LB	1,879	454	22,666 (8)	24,999
AND						
1005-1103 (10)	STEEL BEAM BEARING PILES, HP 12 x 53	LF	943	590	---	1,533
AND						
1005-1153 (10)	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 x 53	EA	42	24	---	66
AND						
1019-0050 (11)	PROTECTIVE COATING FOR REINFORCED CONCRETE SURFACES (PENETRATING SEALERS, BRIDGE SUPERSTRUCTURE)	SY	---	---	690	690

- (1) ITEMS IN BRIDGE STRUCTURE LUMP SUM ITEM 8030-0001 GIVEN FOR INFORMATION ONLY
- (2) INCLUDES CLASS AAAP CONCRETE IN DECK SLAB AND APPROXIMATELY * CUBIC YARDS OF CLASS AAAP CONCRETE TO ACCOUNT FOR STAY-IN-PLACE FORM TROUGHS
- (3) INCLUDES CLASS AA CONCRETE IN SHEAR BLOCKS, ABUTMENT BACKWALLS, CHEEKWALLS AND UWINGS ABOVE THE HORIZONTAL CONSTRUCTION JOINT NEAR THE BRIDGE SEAT
- (4) INCLUDES CLASS AA CONCRETE IN CURBS, BARRIERS, SIDEWALKS, DIVISORS AND CONCRETE DIAPHRAGMS
- (5) QUANTITY TO BE USED FOR CLASS AA CONCRETE UNDER THE DECK COLUMN HEADING ON THE STRUCTURE COST DATA FORM
- (6) IF PILE TIP REINFORCEMENT IS REQUIRED, SHOW FOLLOWING NOTE ON DRAWING:
INCLUDE * PILE TIP REINFORCEMENT
(*SHOW NUMBER AND SPECIFY TYPE [NORMAL OR HEAVY DUTY] OF PILE TIP REINFORCEMENT REQUIRED)
- (7) SHOW NUMBER AND LENGTH OF TEST PILES ON THE DRAWING (i.e., 2 @ 45 ft., 1 @ 45 ft. or 1 @ 30 ft.)
- (8) INCLUDES * LBS. OF EPOXY-COATED DOWELS
(*SHOW QUANTITY OF EPOXY-COATED DOWELS)
- (9) FOR AS DESIGNED STRUCTURE, INCLUDED IN BRIDGE BID ITEMS. FOR ALTERNATE DESIGNS, INCLUDED IN BRIDGE STRUCTURE LUMP SUM BID ITEM
- (10) INCLUDED IN BRIDGE BID ITEMS
- (11) ONLY APPLY PROTECTIVE COATING IF CONCRETE IS POURED BETWEEN SEPTEMBER 1 AND MARCH 1 PER PUBLICATION 408M, SECTION 1001.3(k)6

Figure 1.6.4.11-2 – Tabulation of Bridge Bid Items and Approximate Quantities (continued)

TABULATION OF STRUCTURE ITEMS			
ITEM NO.	DESCRIPTION	UNIT	TOTAL
8210-0001	DESIGN OF BRIDGE STRUCTURE (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX	LS	LUMP SUM
EITHER			
8250-0001	CONSTRUCTION OF PRESTRESSED CONCRETE STRUCTURE, S-XXXXX	LS	LUMP SUM
OR			
8251-0001	CONSTRUCTION OF STEEL STRUCTURE, S-XXXXX	LS	LUMP SUM

TABULATION OF STRUCTURE ITEMS			
ITEM NO.	DESCRIPTION	UNIT	TOTAL
8213-0001	DESIGN OF RETAINING WALL (NO AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX	LS	LUMP SUM
EITHER			
8255-0001	CONSTRUCTION OF CONCRETE RETAINING WALL, S-XXXXX	LS	LUMP SUM
OR			
8256-0001	CONSTRUCTION OF PREFABRICATED RETAINING WALL, S-XXXXX	LS	LUMP SUM

TABULATION OF STRUCTURE ITEMS			
ITEM NO.	DESCRIPTION	UNIT	TOTAL
8216-0001	DESIGN OF REHABILITATION AND/OR WIDENING (AS-DESIGNED FOUNDATION PROVIDED), S-XXXXX	LS	LUMP SUM
8260-0001	CONSTRUCTION OF REHABILITATION AND/OR WIDENING, S-XXXXX	LS	LUMP SUM

Figure 1.6.4.11-3 – Tabulation of Structure Bid Items for Design-Build Projects

1.6.5 Stake-Out Sketch

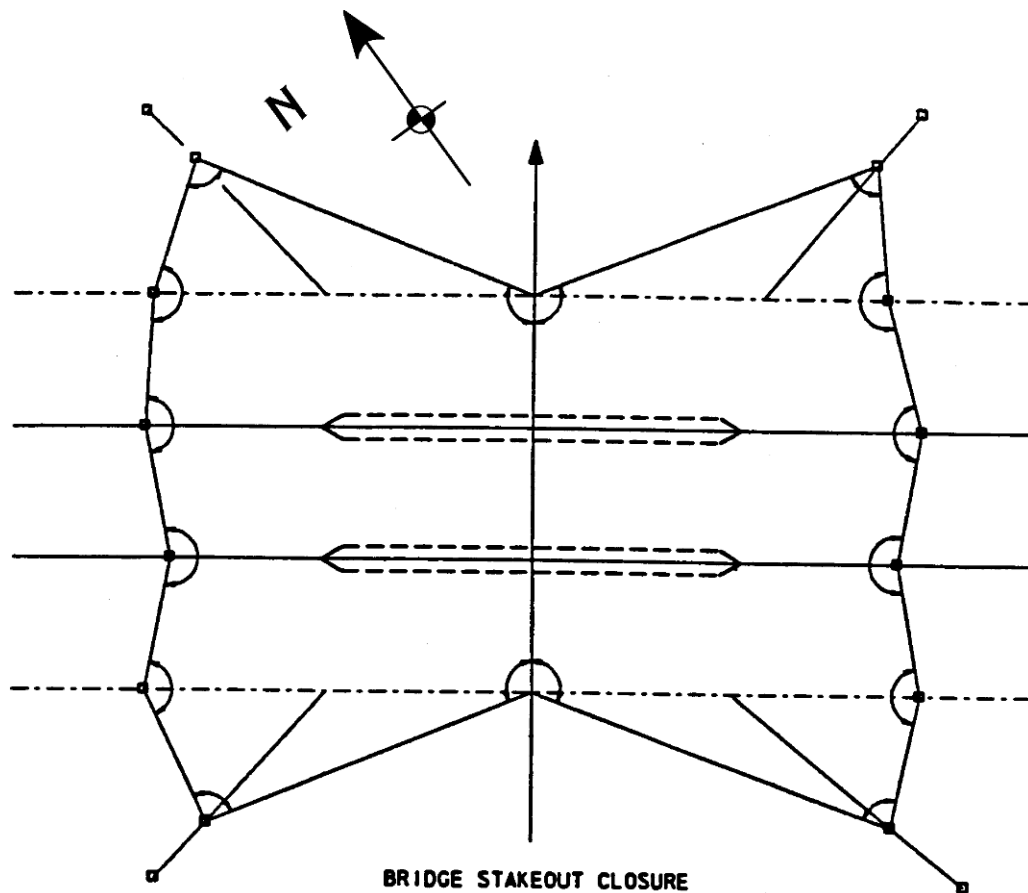
- (a) A stake-out sketch shall be shown, preferably on the first or second sheet of the structure drawings. There should be ample open space outside of the sketch to allow wing and barrier line extensions for stake point recordings. The sketch need not be to scale. Frequently, exaggerations of curvature, angle, etc., are necessary to show the information clearly.
- (b) The sketch shall be as simple as possible, but as complete as possible so that the structures will be constructed according to the plans.
- (c) All necessary tie-in dimensions between highway alignment, working points, lines of structure, and other control points shall be shown in feet to two decimal places on the sketch.
- (d) Provide a table of coordinates for all workpoints. Provide a table of coordinates for the baseline. Provide coordinates to four decimal points and add the following note: Four place coordinates are for computational purposes only and do not imply a precision beyond two decimal points.

1.6.5.1 Stake-Out Guidelines

- (a) The stake-out shall be referenced to one straight base line, except in the case of dual structures, where two straight base lines, properly referenced to each other, can be used. The base line will be the centerline of the highway (if tangent), or the long chord connecting the points where the centerline of the highway intersects the face of the abutments on a curved highway, or the tangent line at the point of intersection of highways or the highway and a stream or river. Generally, dimensioning along the long chord is preferred on sketches for viaducts with a long series of spans. In special situations, some other base line can be used if particularly convenient.
- (b) The sketch shall show the base line and the shape of the exterior face of the substructure (abutments and wingwalls). All corners shall be referenced by showing work points and distances to the base line. Wingwall angles to the front face of abutments shall also be referenced. Work point coordinates may be shown on the plan.
- (c) At intermediate piers, the skew angle between the centerline of the pier and the base line is required. The location of the intersection of pier centerline with base line shall be tied to other parts of the substructure by base line dimensions. The distance from the base line to the centerline of roadway along the centerline of the pier shall be given. The station of the intersection points at the base line shall be shown. Distances between the outside faces of each barrier shall be shown.
- (d) For multi-level structures, each level shall be sketched separately, but referenced to the same base line.
- (e) The stake-outs for box culverts shall include inside faces of walls, ends of the culvert, and the front face of the wingwalls. Reinforced concrete arch culverts and metal culverts shall be treated similarly.

1.6.5.2 Procedure to Ensure Against Discrepancies in Bridge Stake-Out

- (a) The structure stake-out sketch and reference stake locations shall be recorded in a Department survey notebook.
- (b) Original stake-out field notes shall be recorded in a survey notebook. When other copies are required, this information shall be taken from the original survey notebook.
- (c) An error of closure on the stake-out shall be recorded in the survey notebook. This error of closure shall reflect a comparison between measured and computed angles and distances of a traverse around the near line of the offset stakes of all working lines and shall meet the minimum error of closure of one part in 10 000 (see Figure 1.6.5.2-1).
- (d) A complete centerline tie shall be made at the ends of a structure to ensure proper location.



Run a closed traverse around the near line of offset stakes of all working lines. On projects where various phase stakeout is required, run the intermediate traverse as the stake-out progresses.

Upon completion of an acceptable closure of the perimeter traverse of 1 part in 10 000, compute the individual closures between each pier and abutment lines. These computations may be made in the field by the party Chief or in the office by a member of the District Bridge Unit.

Record this sketch and closure in a survey field book to be made readily available for inspection.

Figure 1.6.5.2-1 – Sections at End and Beginning of Structure

1.7 GENERAL NOTES

From the following list, notes related to the type of structure shall be shown on the general plan of the drawings. The notes applicable to specific elements of a structure may be shown on the corresponding sheets. Special notes shall be written, or these notes revised, to meet special conditions on individual projects. Use the imperative mood in writing general notes on the bridge plans.

1.7.1 Design Specifications

1. AASHTO LRFD Bridge Design Specifications*, and as supplemented by Design Manual, Part 4 (including latest revisions**). (Also, include applicable specifications as indicated in PP1.1.1.)

*specify year, i.e., 2010

**specify month and year

2. Live load distribution to girders is based upon ____ method (see PP1.7.10, Instruction 1).
3. Design is in accordance with the LRFD method.

1.7.2 Design Live Loads

1. PHL-93 or P-82 (204 kip permit load)
2. Fatigue design is based on the following:

Steel structures: ADTT ____ (year)
 (one-directional)

Prestressed concrete: ADTT ____ (year)
 (one-directional)

3. Maximum allowable tensile stress in precompressed tensile zone:

$$0.0948\sqrt{f'_c}$$

1.7.3 Dead Loads

1. Includes surface area density of 0.030 ksf for future wearing surface on the deck slab (see PP1.7.10, Instruction 2).
2. Includes a surface area density of 0.015 ksf for permanent metal deck forms which takes into account the weight of the form, plus the weight of the concrete in the valleys of the forms (see PP1.7.10, Instruction 3).
3. Includes ____ kip/ft for utilities, ____ kip/ft for inspection walk and ____ kip/ft for sound barrier.

1.7.4 General

1. Provide materials and perform work in accordance with Specifications, Publication 408*, AASHTO/AWS D1.5M/D1.5** Bridge Welding Code, and contract special provisions.

*Specify year, i.e., 2011

**Specify applicable year, i.e., 2008 (Use AASHTO/AWS D1.1/D1.1M:2008 for welding not covered in AASHTO/AWS D1.5M/D1.5: 2008). Designers to verify date to be consistent with Pub 408 1105.03 (m).

2. Provide structural steel conforming to AASHTO *** (ASTM ****) designation, except when noted otherwise.

***Specify applicable number, i.e., M 270/M 270M

****Specify ASTM designation, i.e., A 709/A 709M

3. Provide 2 in. concrete cover on reinforcement bars, except as noted.
4. Use Class AAAP cement concrete in deck slab, sidewalks and top slab of cast-in-place concrete box culverts at grade.
Use Class AAA cement concrete in precast channel beams
Use Class AA cement concrete in curbs, barriers, divisors, concrete diaphragms, abutment backwalls, cheek walls, shear blocks, U-wings above bridge seat construction joint, footings (when specified) and sound barriers.
Use Class A cement concrete in piers, abutments below bridge seat, pedestals, wingwalls, retaining walls, footings, arch culverts, spandrel walls, walls and top and bottom slabs of cast-in-place concrete box culverts under fill, walls and bottom slab of cast-in-place concrete box culverts at grade and caissons.
Use Class C cement concrete below the bottom of footings when specified.
5. A higher class concrete may be substituted for a lower class concrete at no additional cost to the Department, if approved by the District Bridge Engineer.
6. Provide Grade 60 reinforcing steel bars that meet the requirements of ASTM A 615/A 615M, A 996/A 996M or A 706/A 706M. Do not weld Grade 60 reinforcing steel bars unless specified. Grade 40 reinforcing steel bars may be substituted with a proportional increase in cross-sectional area, if approved by the Chief Bridge Engineer. Do not use rail steel A 996/A 996M reinforcement bars in bridge piers, abutments, shear blocks, beams, footings, piles, barriers or where bending or welding of the reinforcement bars is indicated.
7. Use epoxy-coated reinforcement bars in the deck slab, barriers, sidewalk, abutment backwalls, U-wings above the construction joint, stirrups protruding from diaphragms and prestressed beams into the deck slab, and pier and abutment seat bars where expansion dams are used. Epoxy-coat J-bar reinforcement in substructure units and epoxy-coat other substructure reinforcement bars as indicated.
8. Galvanized reinforcing steel bars may be substituted for epoxy-coated reinforcing steel bars at no additional cost to the Department.
9. Rake-finish all horizontal construction joints, except as indicated.
10. Site Class Class E. (*Specify either "is" or "is not")
11. Use retarder admixture conforming to Publication 408 (specify year) in the concrete deck slab (see PP1.7.10, Instruction 4).
12. Verify all dimensions and geometry of the existing structure in the field as necessary for proper fit of the proposed construction (see PP1.7.10, Instruction 5).
13. Construct deck slab transverse construction joints parallel to bridge centerline of bearings.
14. Abutment backwalls may be placed up to a construction joint below the level of the bottom of deck slab prior to construction of the deck.
15. Notify the regional headquarters of the Fish Commission prior to construction and cooperate with Fish Commission during construction. (Include name, address and telephone number for Waterway Conservation Officers)
16. Place cheekwall, concrete shear blocks, and backwall concrete after beams are set in position.
17. Chamfer exposed concrete edges 1 in. by 1 in., except as noted.
18. All dimensions shown are horizontal, except as noted.

19. Use either permanent metal forms or removable forms to construct the deck slab. (Specify type of forms if required by design conditions.)
20. Deck slab thickness includes a ½ in. integral wearing surface.
21. Superstructure dimensions shown are for a normal temperature of 68° F.
22. Before driving piles, place and compact, to footing elevation, specially selected material which contains no rock to interfere with pile driving. Auguring or pre-boring will be permitted to the original ground. (Note: to be used if footing is above existing groundline.)
23. Spread footings may be ordered by the Engineer to be at any elevation or of any dimensions necessary to provide a proper foundation. (Not applicable for footing set on piles.)
24. Use corrosion inhibiting admixture in the concrete deck slab (see PP1.7.10, Instruction 8).
25. Provide minimum embedment and splice lengths in accordance with Standard Drawing BC-736M, unless otherwise indicated.
26. Prepare bearing areas as specified in Publication 408, section 1001.3(k)9.

1.7.4.1 Notes for Proprietary Walls (For plans prepared by a proprietor for the Contractor)

1. These drawings are intended for use only at the site for which they are prepared. *_____ disclaims any liability for any other use (*write company name).
2. For additional design information, core borings and other geotechnical information not shown on these plans, refer to the original design plans, S-number_____.
3. All design assumptions are validated through either notes to the Contractor or details on these drawings.

1.7.4.2 Notes for Bridge Rehabilitation Plans

1. Do not consider any of the data on the existing structure supplied in the original design drawings or made available to you by the Department or its authorized agents as positive representations of any of the conditions that you will encounter in the field.
2. The information shown on the plans for the existing bridge is not part of the plans, proposal, or contract and is not to be considered a basis for computation of the unit prices used for bidding purposes. There is no expressed or implied agreement that information is correctly shown. The bidder is not to rely on this information, but is to assume the possibility that conditions affecting the cost and/or quantities of work to be performed may differ from those indicated. (List the original design drawings and any rehabilitation drawings, if applicable. Indicate the approval or recommended date in parenthesis.)
3. The following standard note should appear on each of the existing bridge plans which are to be supplied to the contractors:

Bidders are advised to field-verify information presented. The data shown herein is not a part of the plans, proposal, or contract, and is not to be considered as a basis for computation for any purpose.

4. After obtaining the laboratory tests results, the following standard note should appear on all bridge painting, demolition, and rehabilitation projects:

The existing bridge structural members contain (Or do not contain) lead paint and other toxic materials {such as cadmium, chromium, arsenic, etc. (Show this detail based on the laboratory test results)} based on laboratory testing.

1.7.5 Notes for Piles**1.7.5.1 Notes for Pile Driving Requirements**

The applicable driving method shall be specified in the foundation submission and shown in the general notes on the design drawings. Driving shall be controlled by the wave equation.

- (a) Method A - Use when bearing piles are driven to absolute refusal

Control pile driving by the wave equation analysis. Drive test piles to absolute refusal. The Engineer shall verify, from the test pile driving results, the capability of the pile hammer selected by the Contractor. Drive bearing piles to absolute refusal into the stratum defined by a tip elevation which is predetermined by the Engineer from test piles. The Engineer shall determine the acceptability of the bearing piles which attain absolute refusal above the predetermined tip elevations.

- (b) Method B - Use when bearing piles are driven to a capacity determined by the wave equation, but to less than absolute refusal.

Drive test piles to absolute refusal unless otherwise directed by the Engineer. The Engineer shall verify, from the test pile driving results, the capability of the pile hammer selected by the Contractor. Drive bearing piles to a tip elevation and a driving resistance predetermined by the Engineer from a wave equation analysis of the test piles. The Engineer shall determine the acceptability of the bearing piles which attain absolute refusal above the predetermined tip elevations.

- (c) Method C - Use when bearing piles are driven to a capacity based on a pile load test result.

- (1) Control pile driving by the wave equation analysis.

- (2) Drive test piles to absolute refusal unless otherwise directed by the Engineer. The Engineer shall verify from the test pile driving results the capability of the pile hammer selected by the Contractor.

- (3) Drive test piles at ____ before driving the load test piles at ____ (indicate locations). Drive all test piles relative to the pile load test first in order to determine the most representative location for the pile load test.

- (4) The pile load test at _____ is intended to be representative of the bearing piles at _____ (indicate locations).

- (5) Pile load tests at _____ may be conducted concurrently. (Use when there is no need to conduct load tests in consecutive order).

- (6) Drive load test piles to a driving resistance and/or a tip elevation predetermined by the Engineer from a wave equation analysis of the test piles.

- (7) Apply the load for the pile load test not less than ____ days after the test load pile has been driven.

- (8) Do not drive bearing piles before the representative pile load test is completed and the results are evaluated by the Engineer.

- (9) Drive bearing piles to a tip elevation and a driving resistance predetermined by the Engineer from the pile load tests. The Engineer shall determine the acceptability of the bearing piles which attain absolute refusal above the predetermined tip elevations.

- (10) Show the following pile load test data on the drawings (modify as required):

Pile Load Test Data

In quantities:

Quick pile load test (*) _____ (each)
 Additional quick pile load test (**) _____ (each)

In or near general notes:

Pile load test data:

Location (***)	XX
Type and size of pile	XX
Design load	XX
Estimated length	XX
Minimum required resistance factor (ϕ)	XX
Minimum required ultimate bearing capacity	XX
Minimum required capacity of test equipment (****)	XX

(*) The load test will be the quick type unless otherwise specified in the foundation approval or recommendation.

(**) Include additional pile load test when specified in foundation approval.

(***) Identify by substructure unit.

(****) Equate to nominal structural capacity of the pile, determined as follows:

For H and unfilled tubular steel piles:

$$P_u = A_s f_y$$

For CIP piles:

$$P_u = 0.85 \bar{f}_c (A - A_s) + A_s f_y$$

where:

P_u = ultimate structural capacity (kips)

A = gross area of concrete (in²)

A_s = area of steel H or tubular piles, or thick wall shell or rebars for cast-in-place piles (Exclude corrosion allowances and area of thin wall shells)

$\bar{f}_c$ = Compressive structural design strength of concrete at 28 days. (ksi)

f_y = specified minimum yield strength of steel (ksi)

These ultimates are only for the purpose of sizing the load test equipment for laterally supported piles with zero eccentricity, and are not applicable to the structural design of the piles. Where it is obvious that the soil will develop significantly less than the ultimate structural capacity of the pile, the test equipment should be sized to ensure a plunging failure of the pile based on soil strength parameters.

1.7.5.2 Notes for Pile Installation Information

The following table is to be included on the General Notes sheet of the bridge plan and is to be completed for the test piles after installation on the “as-built” plans:

Substructure Unit	Pile Type	Pile Tip (None/Normal/Heavy Duty)	Pile Tip Elevation	Factored Design Load (kips)	Ultimate Pile Capacity at End of Driving (kips)	WEAP or PDA

1.7.6 Notes for Reinforced Concrete Box and Arch Culverts

Do not exceed a 2 ft. difference in fill elevation on the sides during placement of the backfill. Do not allow the wheels of rollers to come closer than 1 ft. to the face of the structure during compaction of the backfill.

1.7.7 Notes for Steel Beams and Girders

1. If beams (girders) cannot be shipped in the lengths shown on the plans, field splice(s) will be permitted at the request of the Contractor, but no compensation will be allowed for the splices (see PP1.7.10, Instruction 6).
2. If beams (girders) can be fabricated in lengths longer than the sections shown on the plans by eliminating field splices, field splice(s) may be omitted at the request of the Contractor. The Contractor assumes full responsibility for securing a hauling permit. Approval for elimination of a field splice at the shop drawing stage does not obligate the Department to issue a hauling permit (see PP1.7.10, Instruction 7).
3. Do not use form support systems that will cause unacceptable overstress or deformation to permanent bridge members.
4. All fasteners are 7/8 in. diameter HS bolts, except as noted.
5. Ream subdrilled or subpunched holes for field splices in the fabrication shop.
6. Prepare bearing areas as specified in Publication 408, Section 1001.3(k)9.
7. Do not make welds by manual shielded metal arc process for primary girder welds, such as flange-to-web welds or for shop splices of webs and flanges.
8. Do not weld permanent metal deck forms or other attachments to girder top flanges in tension areas. (Tension areas of top flanges are designated on the plans.) Threaded studs for the support of the overhang deck forming bracket is permitted provided the threaded stud is attached with the same welding processing as the shear studs.
9. Welding of reinforcement bars during fabrication or construction is not permitted unless specified.
10. Provide welded stud shear connectors manufactured from steel conforming to ASTM A 108.
11. Set anchor bolts to template or in preformed holes. Do not drill unless specifically indicated on plans. Fill the preformed holes with non-shrink grout. Fill the clearance between anchor bolts and holes in masonry plates with approved non-hardening caulking compound conforming to Publication 408, Section 705.8.
12. Paint structural steel in accordance with Publication 408, Section 1060.
13. Fabricate all members or member components designated as fracture-critical members (FCM) to conform to the requirements of Design Manual, Part 4, Section 6.6.2, and AASHTO LRFD Bridge Design Specifications, Article 6.6.2, and Publication 408, Sections 1105.02(a)5 and 1105.03(m)9. Meet the base metal Charpy V-notch (CVN) requirements for Zone 2.

14. Metallize structural steel in accordance with the special provision - Shop Metallizing and Painting of New Structural Steel. The flange, bearing stiffener plates and splice plates indicated are oversized in width to accommodate the reduction due to edge grinding.
15. Stability of partial girders and complete girders is to be maintained by the Contractor during erection, until all girders and diaphragms are in-place and all bolts are properly installed. Erection loads including self weight of the steel members, wind loading and construction live load effects are to be evaluated by the contractor for stability, stresses and deflections on the steel members during any stage of erection.
16. An alternate slab placement sequence may be permitted at the request of the Contractor. Submit for review and approval to the Department a revised slab placement sequence with support calculations and computer stress analysis. Satisfy the requirements of the original slab placement sequence. Obtain written approval prior to the use of the revised slab placement sequence and/or camber values. No compensation will be allowed for the development and approval of the revised slab placement sequence and camber values. The Department will be the sole judge of the acceptability of the revised slab placement sequence and camber values.
17. Consultant to specify if heat-curved beams (girders) are (are not) permitted.
18. See D6.7.2, D6.7.2.2.P, D6.10.3.2.5.2P, and D6.13.2.8 for additional notes to be shown on contract drawings.

1.7.8 Welding Notes for Rehabilitation of Structures or where Field Welding is Permitted

1. Welding specifications: AASHTO/AWS D1.5M/D1.5 Bridge Welding Code (specify year) consistent with Pub 408 1105.03(m) and the contract special provisions. Do not field-weld on any part of the existing bridge, except where shown on the drawings, without prior approval of the Engineer.
2. Welding of existing structural steel: Use the shielded metal arc process and low hydrogen electrodes which are compatible with the base metal as specified, and in accordance with an approved Weld Procedure Specification.
3. Make tack welds with the same type of electrode and incorporate in the final weld. No other tack welding will be permitted.
4. Do not weld when surfaces to be welded are moist or exposed to rain, snow, or wind, or when welders are exposed to inclement conditions that will adversely affect the quality of the work.
5. Do not weld or burn when the temperature is below 0° F. Preheat and maintain the temperature of the metal to at least 70° F when the temperature of the metal is between 0° F and 32° F during welding or burning.
6. Preheat the steel to the specified minimum temperature for a distance equal to the thickness of the part being welded, but not less than 3 in. in all directions from the point of welding.
7. Remove by application of heat any moisture present at point of weld. Provide windbreaks for protection from direct wind.
8. Prior to placing the weld, thoroughly clean all portions of new and existing surfaces to receive welds of all foreign matter, including paint film, for a distance of 2 in. from each side of the outside lines of the weld.
9. Test completed welds using visual and nondestructive methods in accordance with AASHTO/AWS D1.5m/D1.5 Bridge Welding Code Chapter 6.

1.7.9 Utility Notes

Coordinate, locate, and conduct all work related to public and private utilities in accordance with Publication 408, Sections 105.06 and 107.12

1.7.10 Instructions

1. Indicate either DM-4 distribution factors, grid analysis method, three-dimensional finite element analysis method, or other.
2. The additional dead load shall not be applied to structures under fill.
3. Metal deck forms with blocked out valleys are commercially available and may be specified when economical for rehabilitation projects or when dead load is a control. Removable forms may also be specified for each case.
4. The note shall be shown on the drawing when retarder admixture is necessary in the concrete deck slab for conditions other than temperature control (mostly in skewed continuous structures).
5. The note shall appear on the plans for repairing, rehabilitating, widening, or extending the existing structures.
6. The design details/splices shall be coordinated with PP1.13, "Hauling Restrictions and Permits", and the standard note adjusted accordingly. If needed, the special provisions shall be expanded or adjusted by the designer. Optional field splice details should not be shown, but the note shall be shown on the drawing for beams or girders between 70 ft. and the limiting lengths specified in PP1.13.2.
7. The design details/splices shall be coordinated with PP1.13 and the standard note adjusted accordingly. If needed, the special provisions shall be expanded or adjusted by the designer. For beams or girders exceeding the lengths specified in PP1.13.2, field splice details shall be shown, and the note shall appear on the plans.
8. The note shall appear on the drawing when corrosion inhibiting admixture is to be specified for the project with the approval of the Chief Bridge Engineer. Indicate the time to corrosion.

1.7.11 Timber Notes

At the completion of preservative treatment, clean the treated material by a post-treatment steaming specified for the individual type of material or species. (Use this note for wooden bridges.)

1.7.12 Mechanically Stabilized Earth Walls

I hereby certify that all design assumptions have been validated either through construction details or notes on these drawings or through the contract plans and special provisions. (Show this note above the P.E. seal on structure plan.)

1.7.13 Additional Notes to be shown on Contract Drawings

For additional notes to be shown on contract drawings, see Figure PP1.6.4.11-2 reference (6) for Pile Tip Reinforcement, PP1.9.4.3.1(b)(5) for Geotechnical notes, PP1.10.1.1 for Alternate Design, PP1.10.6.2.2 for Plan Revisions, PP3.3.4.7(l) for Prefabricated Retaining Walls, PP3.4.2(a) for Concrete Deck Sealant, D10.9.1.9P for Micropile Notes, D11.13P3 for Gabion Retaining Walls, and D14.7.6.3.5 for bearing lift-off condition.

1.8 DESIGN COMPUTATIONS**1.8.1 General Requirements**

- (a) Original design computations shall be made in such a way that they can be microfilmed and will produce clear and legible copies. The minimum font size shall be 10 pt. Computations shall be arranged so that a Reviewer may easily follow the subject and procedures of the design analysis. Each sheet of design computations shall be thoroughly checked and initialed by the designer and the checker. An index sheet shall be provided for easy reference. The computations shall be of good contrast and shall be only on one side of 8 ½" x 11" (U.S. Customary Letter Size) sheets for ease of microfilming. An exception to the minimum font size may be approved by the District on a case-by-case basis if space is limited and all caps are used. For these limited cases, a minimum 8.5 pt size in a font such as ARIAL may be considered.

- (b) Design computations are not required for any portion of a structure for which the information is taken verbatim from an applicable current Department standard. In such cases, the standard shall be referenced in the calculations.
- (c) In general, the design computations shall consist of these items: geometry calculations, structural analysis, quantity computations and necessary sketches.

Each phase of computer-generated computations should include a table showing the summary of results unless the critical items are obvious, e.g., for "Structural Analysis" the table showing the summary of results shall indicate the actual and allowable stresses for various loading groups.

Preliminary computations are not required, but may be included in the computations if identified and separated from the final design computations.

When methods or formulas which are not in general use are employed, the source shall be given, including title and edition of the book, name of author(s), publisher and page numbers.

- (d) Title sheet and cover of original design computations shall show the name of county, project route (S. R. number and section number), station of structure, S-number of the drawings, and ECMS Project number, and be signed and sealed by the responsible engineers. Each sheet of calculations shall be dated by designer and checker.
- (e) Original design computations shall be bound in hard covers that completely enclose the parts of the fasteners used. Bindings with exposed fasteners shall not be used. Upon final acceptance of the drawings, the original design computations shall be submitted to the District Office as a permanent record.

1.8.1.1 Policy for Designer's Responsibility for Constructibility

CASE A: Construction Methods and/or Sequences Specified by the Designer in the Contract Documents

When the design model assumes that the design loads will be applied in a certain sequence or under certain conditions, the designer must provide enough information in the contract documents (including plans) to ensure that construction sequence and/or methods will be consistent with the design assumption. In such cases, the designer is fully responsible and accountable for constructibility due to all temporary and cumulative design loads which eventually become part of the final design loads.

CASE B: Designs Based on New Design Methods which May Affect the Constructibility of a Common Bridge by Decreasing the Size of its Members. Optimized Designs are also Included in Case B.

The Designer is responsible for providing an appropriate notice in the contract documents (including plans) if typical construction methods used successfully in the past will not be permitted because of higher temporary construction stresses that may occur due to design dead loads and/or construction loads. As an alternative, the designer may design for a typical construction method which has been used successfully in the past, if it is economical to do so, and indicate the assumed construction method in the contract documents. The reason for assigning this responsibility to the designer is that the designer is in a better position to be aware of the affect that changes in design specifications and design methods will have on constructibility than would be the Contractor.

Although this policy assigns responsibility to the designer for Case B, the degree of accountability will vary with the circumstances as follows:

- (1) If new design specifications or design methods implemented by the Department contain guidelines and/or commentaries which identify potential problems in constructibility, the designer will be held accountable if constructibility problems occur because of a failure to introduce constructibility guidelines in the design.
- (2) The absence of constructibility guidelines or commentaries in new design specifications will not relieve the designer of responsibility for addressing constructibility. Accountability, should problems occur, will depend on whether the designer made a reasonable effort to address constructibility. An example of a reasonable effort in this case would be a request for approval of constructibility criteria during design, preferably at TS&L stage.

For contractor-designed alternates and Design-Build projects, the Contractor is completely responsible and accountable for constructibility.

Even with reasonable efforts, some constructibility problems may still occur because of the current state-of-the-art. In such cases, the Department, in the past, has been quite liberal in assuming some degree of accountability and will continue to do so.

The Designer's responsibility for constructibility must be considered in the technical scope of work in consultant engineering proposals.

For bridges under construction, the revised design specifications can often be met by modifying the size and sequence of the deck pours.

1.8.2 Computer Programs

When computations are performed using a computer program, the input and output, with an explanation of terms, and the assumptions and computations used for the determination of the input values, shall accompany the design computations.

For computer programs not available through the Department, a sketch with an explanation of all abbreviations and symbols shall accompany the input and output sheets of the program. All computer programs shall be identified. Design methodology employed by the programs shall be indicated: line girder, grid analysis, finite element, etc.

For software review requirements, see PP1.4.5.

1.8.3 Rating Computations

- (a) Computations shall be made showing the inventory and operating ratings in terms of H, HS, ML, TK-527 and PHL-93. Computations shall also be made showing the operating rating for P-82 vehicle. The ratings shall be calculated using the loadings described in A3.6 and D3.6. For steel and prestressed girders, concrete box culverts, steel floorbeams and steel trusses, Tables D3.4.1.1P-1, D3.4.1.1P-2, D3.4.1.1P-4, D3.4.1.1P-5 and D3.4.1.1P-6, respectively, shows the limit state load combination corresponding to the different rating vehicles. The original rating computations are to be included with the original design computations. A description of the method of analysis is to be included in the calculations. The ratings shall be determined by the design method and analysis method (DM-4 distribution factors or a refined method) which were used in design, unless changed by direction of the Chief Bridge Engineer.

Rating calculations shall clearly show the total inventory and operating capacity and the live load capacity, so that the calculations can be used throughout the life of the structure.

- (b) The bridge plans for deck replacement, overlays, major rehabilitation and new construction, including Contractor's alternate designed bridges, shall show the bridge load ratings as indicated in the sample chart below:

Table 1.8.3-1 – Bridge Load Ratings

ADTT (at the Time of Design)
Cumulative ADTT (at the Time of Rehabilitation)

Span No.		Beam Type and Size					
		H 20	HS 20	ML-80	TK-527	PHL-93	P-82
Inventory Rating (IR)	Distribution Factor	0.720	0.720	0.720	0.720	0.720	---
	Location	CL	CL	CL	CL	CL	---
	Limit State	SERV III	SERV III	SERV III	SERV III	SERV III	---
	Rating Factor	1.39 M	1.19 M	1.07 M	1.02M	1.04 M	
Operating Rating (OR)	Distribution Factor	1.018	1.018	1.018	1.018	1.018	1.018
	Location	0.05 L	0.45 L	0.45 L	0.45L	0.35 L	0.25 L
	Limit State	STR II	STR II	STR II	STR II	STR IA	STR II
	Rating Factor	2.75 S	2.17 S	1.96 S	1.90 S	2.09 S	1.00 S

Critical Member Interior

Maximum Factored Flexural Resistance (kip-ft) 13,269 k-ft

Location 0.45 L

Maximum Factored Shear Resistance (kips) 153 kips

Location 0.45L

Notes: “M”, “S”, and “SM” denote that moment, shear, and shear/moment interaction control the rating factor, respectively. Given distribution factor is the vehicular load distribution factor used to produce the given rating. For the STR-IP limit state, the vehicular live load distribution factor accounts for the presence of pedestrian loads, if applicable.

Include with the chart on the bridge plans the following information:

- (1) The force effect controlling the rating.
- (2) The limit state used to obtain each of the ratings and each critical moment and shear value.
- (3) Live load distribution factors for shear or moment for each load combination used to produce that rating.
- (4) IR = Inventory Rating, OR = Operating Rating, ML = Pennsylvania Maximum Legal Load (ML-80), P-82 = Pennsylvania Permit Load
- (5) For multiple span structures, identify and provide data for critical span(s) only.
- (6) Provide moment and shear influence lines for the structure that is designed using a refined method of analysis.
- (7) Identify critical (in moment or shear) member(s) and location.
- (8) Load due to future wearing surface is included in the ratings. (The engineering districts may require the rating value with and without future wearing surface.)
- (9) The critical moment and shear shall be determined from the operating rating calculations.
- (10) Identify whether shear or moment controlled the rating for each rating value.

(11) Identify beam properties used.

(12) Identify continuity assumption used (i.e., whether positive moment reinforcement was considered in ratings).

The table may be modified in order to provide all of the required information. All rating information should be shown together on the first sheet of the bridge plans or indexed on the first sheet.

1.9 BRIDGE SUBMISSIONS – DESIGN PHASE

Refer to Tables 1.9-1, 1.9-2 and 1.9-3 for determining review and approval responsibility for each bridge submission.

1.9.1 General

The chronology of the bridge-related submissions for approval shall be made as follows:

- (a) Hydraulics and Hydrologic Report (if applicable) (see PP1.9.2)
- (b) Type, Size and Location (TS&L) (see PP1.9.3)
- (c) Foundation Submission (see PP1.9.4)
- (d) Final Review of Plans (see PP1.9.5)
- (e) Final Plans (see PP1.9.6)
- (f) Plans, Specifications and Estimate (PS&E) (see PP1.9.7)

All submissions must include pertinent Q/A forms (refer to Appendix A) without which the submission will be returned without any action by the approving office.

Table 1.9-1 – Review and Approval Responsibility for Federal Oversight Project

FEDERAL OVERSIGHT PROJECTS (7)

(Interstate projects with project cost > \$3 million, other NHS projects costing ≥ \$15 million, and Appalachian Development Highway System Corridor Completion Projects) in accordance with the June 2007 Stewardship and Oversight Agreement

Item No.	Category	TS & L Approval (1)			Final Plans (3) (6)			Remarks
		FHWA	BDTD	District Br. Engr. (2)	FHWA	BDTD	District Br. Engr. (2)	
1. Interstate Pavement Preservation (PPG) (4)	Bridge cost ≤ \$3 million (8)	-	-	✓	-	-	✓	
	Bridge cost > \$3 million (8)	✓	-	-	-	-	✓	
2. 3R type of work	Deck rehab. ≤ \$15 million (4) (8)	-	-	✓	-	-	✓	
	Deck rehab. > \$15 million (4) (8)	✓	-	-				
	Culvert extensions	-	-	✓	-	-	✓	
	Wall rehabs /extension	-	-	✓	-	-	✓	
	Deck replacement ≤ \$15 million (4) (8)	-	-	✓	-	-	✓ (See remarks)	Final plans for multi-span steel structures, steel structures with skew < 70°, curved girders, and fracture critical structures must be sent to BDTD for approval.
	Deck replacement > \$15 million (4) (8)	✓	-	-				
	Superstructure rehabilitation or replacement ≤ \$15 million (4) (8)	-	-	✓ (See remarks)	-	-	✓ (See remarks)	TS & L and Final plans for multi-span steel structures, steel structures with skew < 70°, curved girders, and fracture critical structures must be sent to BDTD for approval.
	Superstructure rehabilitation or replacement > \$15 million (4) (8)	✓	-	-				
	Substructure rehabilitation ≤ \$15 million (4) (8)	-	-	✓	-	-	✓ (See remarks)	Note that District must submit seismic and fatigue retrofit details to BDTD for approval.
	Substructure rehabilitation > \$15 million (4) (8)	✓	-	-				
3. New Construction or Reconstruction	New interstate bridge cost ≤ \$3 million	-	-	✓	-	-	✓ (See remarks)	Final plans for multi-span steel structures, steel structures with skew < 70°, curved girders, and fracture critical structures must be sent to BDTD for approval.
	New interstate bridge cost > \$3 million	✓	-	-				
	New NHS bridge cost ≤ \$15 million	-	-	✓				
	New NHS bridge cost > \$15 million	✓	-	-				

Table 1.9-1 – Review and Approval Responsibility for Federal Oversight Project (continued)

FEDERAL OVERSIGHT PROJECTS (7)

(Interstate projects with project cost > \$3 million, other NHS projects costing ≥ \$15 million, and Appalachian Development Highway System Corridor Completion Projects) in accordance with the June 2007 Stewardship and Oversight Agreement

Item No.	Category	TS & L Approval (1)			Final Plans (3) (6)			Remarks
		FHWA	BDTD	District Br. Engr. (2)	FHWA	BDTD	District Br. Engr. (2)	
4. Misc. Str. (4)	Culverts	-	-	✓ (2) (See remarks)	-	-	✓ (See remarks)	Culverts under fills greater than 40 ft. must be approved by BDTD.
	Ret. walls < 30 ft. height	-	-	✓ (2)	-	-	✓	TS&L and Final Plan approval for all MSE walls are to be in the District office.
	Ret. walls ≥ 30 ft. height	✓	-	-	-	✓	-	All MSE wall submissions are to be approved in the District office.
	Sound Barrier Walls	-	-	✓	-	-	✓	
	Arches: span ≤ 60 ft.	-	-	✓ (2)	-	-	✓	
	Arches: span > 60 ft.	-	✓	-	-	✓	-	
	Sign Structures	-	-	-	-	-	✓	No formal approval is required.
	DMS Structures	-	-	✓	-	✓	-	
5. Other (4)	Rehab of unusual structures	✓	-	-	-	✓	-	
	All exceptions to HBP program (eligibility) criteria	✓	-	-	-	✓	-	Including deficiency status, sufficiency rating, and 10 year rule
	Design exceptions to structural design criteria and policies	✓	-	-	-	✓	-	Over STRAHNET, coordinate with FHWA for exceptions to 16.0 ft. minimum vertical clearance and changes to clearances currently less than 16.0 ft.. See DM-2, Section 2.20. Other design exceptions must follow guidance as defined in DM-IX, Appendix P and be coordinated with HDD.
	Non-redundant structures	✓	-	-	-	✓	-	
	3-D analysis	✓	-	-	-	✓	-	Final plans approval authority may be transferred to Dist. Br. Engr., depending upon complexity.
	All experimental or demonstration items	✓	-	-	-	✓	-	
	Unusual structures	✓	-	-	-	✓	-	
	Light Poles	-	-	-	-	-	✓	Review and approve for structural capacity only. See Appendix B. (5)

- Notes: (1) Includes Conceptual and Final TS&L submissions for Design-Build projects.
 (2) A copy of approvals must be given to BDTD.
 (3) Scour, safety, expansion dams and load capacity items must be addressed for all work categories.
 (4) Seismic design or assessment must be completed for structures where the work requires removal of the deck, new piers, or new superstructure. For minor maintenance projects, the seismic criteria may be deferred if requested at the TS&L stage.
 (5) Light poles approved under a general submission do not require a re-review and are considered pre-approved for other projects.
 (6) Includes Final Plans for Design-Build projects
 (7) All P.S. & E's for Federal oversight projects are still sent to FHWA for final approval
 (8) Cost is based on TS&L estimate and is a per bridge cost for projects with multiple bridges

Table 1.9-2 – Review and Approval Responsibility for Foundation Approval for all Projects

FOUNDATION APPROVAL FOR ALL PROJECTS (1)

Includes Retaining Walls

Category	FHWA	BDTD(2)	Dist. Br. Engineer	Remarks
1. Footings on bedrock	-	-	✓	
2. Culverts with integral bottom on rock or soil	-	-	✓	
3. Footings on soils	-	-	✓	Including MSE wall footings
4. Foundation on pedestals(must be on rock)	-	-	✓	In mining areas approval is from BDTD
5. Foundation on piles not in mining areas	-	-	✓(3)	
6. Anchored walls	-	✓	(see remarks)	Dist. Br. Engr. may approve anchored walls for local projects.
7. Foundation on piles in mining area, on caissons and on micropiles	✓ Federal Oversight Projects only	✓ PennDOT Oversight & Non-federally funded projects	-	
8. Unusual foundations	✓ Federal Oversight Projects only	✓ PennDOT Oversight & Non-federally funded projects	-	
9. Foundation for arches without ties	✓ Federal Oversight Projects only	✓ PennDOT Oversight & Non-federally funded projects	-	
10. Sound Barrier Walls	-	-	✓	
11. Sign Structure/DMS	-	-	✓	
12. High Mast Lighting	-	-	✓	

- Note: (1) If substructure units of a bridge are supported by different types of foundations, the approval authority will be determined using the most critical foundation type.
- (2) On Federal oversight projects when the TS&L requires FHWA approval, a copy of the foundation approval must be sent to FHWA.
- (3) Structures with a significant amount of friction piles, e.g., major viaducts, major river crossings and > 1,000 ft. of retaining walls, will be submitted to BDTD (and FHWA if Federal Oversight) for approval.

Table 1.9-3 – Review and Approval Responsibility for PennDOT Oversight Projects

PennDOT OVERSIGHT PROJECTS(Interstate projects with cost $\leq$ \$3 million, other NHS projects costing $<$ \$15 million and Off NHS Projects)

in accordance with the June 2007 Stewardship and Oversight Agreement

AND NON-FEDERALLY FUNDED PROJECTS INCLUDING LOCAL PROJECTS

Item/Category	TS & L (1)		Final Plans Approvals (2)		Remarks
	BDTD	District Br. Engr.	BDTD	District Br. Engr.	
1. Deck rehabilitation	-	✓	-	✓	
2. Superstructure rehabilitation	-	✓	-	✓	
3. Substructure rehabilitation	-	✓	-	✓	
4. Culvert extension	-	✓	-	✓	
5. Wall rehabilitation	-	✓	-	✓	
6. Deck & superstructure replacement	-	✓ (see remarks)	-	✓ (see remarks)	BDTD is only responsible for multi-span steel bridges, steel bridges having skew $< 70^\circ$, curved girders, complex and unusual bridges, bridges designed using 3D analysis, and fracture-critical bridges.
7. New bridges & bridge replacements with bridge cost $<$ \$15 million	-	✓ (see remarks)	-	✓ (see remarks)	BDTD is only responsible for multi-span steel bridges, steel bridges having skew $< 70^\circ$, curved girders, complex and unusual bridges, bridges designed using 3D analysis, and fracture-critical bridges.
8. New bridges & bridge replacements with bridge cost $\geq$ \$15 million	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
9. Culverts	-	✓	-	✓	
10. Retaining walls < 30 ft. height	-	✓	-	✓	
11. Arches ≤ 60 ft. Span L	-	✓	-	✓	
12. Mechanically stabilized earth walls	-	✓	-	✓	
13. Sound Barrier Walls	-	✓	-	✓	
14. Retaining walls ≥ 30 ft.	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
15. Arches > 60 ft.	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
16. Designs which use non-AASHTO load distribution factors	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
17. Unusual structures	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
18. Anchored walls	✓	-	✓	-	Final plan approval authority may be transferred to Dist. Br. Engr. on case-by-case basis.
19. <u>All</u> design exceptions to structural design criteria and policies	✓	-	✓	-	Over STRAHNET, coordinate with FHWA for exceptions to 16.0 ft. minimum vertical clearance and changes to clearances currently less than 16.0 ft.. See DM-2, Section 2.20. Other design exceptions must follow guidance as defined in DM-1X, Appendix P and be coordinated with HDD.
20. Non-redundant structures	✓	-	✓	-	
21. <u>All</u> experimental or demonstration items	✓	-	✓	-	
22. Light poles	-	-	-	✓	Review and approve for structural capacity only. See Appendix B.
23. Sign structures	-	-	-	✓	No formal approval is required.
24. DMS structures	-	✓	✓	-	

Note: (1) Includes Conceptual and Final TS&L submissions for Design-Build projects.

(2) Includes Final Plans for Design-Build projects.

A copy of all approvals by Dist. Br. Engr. and completed Q/A forms must be sent to BDTD. Scour, seismic retrofit, safety, expansion dams, and load capacity items must be addressed for all work categories. All design exceptions for structures must be approved by BDTD and/or FHWA. Exceptions or waivers of HBP program eligibility criteria must be approved by FHWA Division.

1.9.2 Permit Applications and Hydrologic and Hydraulic Report

Permit Applications and Hydrologic and Hydraulic Reports shall be prepared to meet the design requirements of Design Manual, Part 2, Chapter 10; Design Manual, Part 4, Chapter PP7; PennDOT Drainage Manual, Publication 584; active Strike-Off-Letters (SOLs); and applicable requirements for regulatory permits.

The following procedures and guidelines apply to all PennDOT projects (Federal-Aid and 100% State) for the submission and review of Hydrologic and Hydraulic Reports:

1. All information required for meeting design and regulatory requirements for individual permits, small projects and General Permit-11 (GP-11) shall be prepared and submitted in electronic copy through the Joint Permit Application (JPA2) system. General Permit-3 (Bank Rehabilitation, Bank Protection, and Gravel Bar Removal) and General Permit-8 (Temporary Road Crossings) may be prepared and submitted electronically if they are a part of a project as an attachment to a GP-11. All other permit applications shall be prepared and submitted in hardcopy. Permits for local projects may be prepared in the JPA2 system but shall be submitted in hardcopy. The information developed shall be reviewed by the District Environmental Manager and District technical staff to ensure compliance with all applicable design, environmental, and regulatory requirements. The information developed shall:
 - a. include the results of the hydrologic and hydraulic analyses associated with design of the waterway structures or encroachments,
 - b. incorporate pertinent prior National Environmental Policy Act (NEPA) documents by reference or excerpt,
 - c. address coordination activities with environmental resources agencies, and
 - d. reflect any commitments or agreements reached which may affect the processing of the permit application.
2. For permit applications prepared and submitted in hardcopy, one copy of the permit application package shall be sent to the Bureau of Project Delivery for quality assurance (Q/A) review. Hydrologic and Hydraulic Reports for municipal structures using Federal-aid funds also shall be submitted by the Engineering District to Bureau of Project Delivery for quality assurance (Q/A) review (see Step 5 below). For permit applications prepared and submitted in hardcopy, the Engineering District shall submit one additional copy of the Hydrologic and Hydraulic Report to the Bureau of Project Delivery for transmittal to the Federal Highway Administration (FHWA) for review and approval of projects involving any of the following issues:
 - a. Significant or controversial channel changes.
 - b. Significant or controversial backwater easements.
 - c. Significant bridge scour (usually manifested by high stream velocity, severe waterway constriction, deep foundation and/or expensive scour mitigation measures).
 - d. Permanent impoundments or causeways involving roadway embankments.
 - e. Major bridges with costs of more than \$15 million.
3. For Joint Permit Applications, the Engineering District shall perform necessary coordination with the applicable counties and municipalities pursuant to 25 Pa.Code §105.13(d)(1)(v-vi) pertaining to the Stormwater Management Act (32 P.S. §§680.1 et seq.) and the Floodplain Management Act (32 P.S. §§679.101 et seq.). For permit applications involving communities with Stormwater Management Plans, or Floodplain Management Plans, implemented under the Acts, the Engineering District shall request from the local municipality a written statement that the proposed project is consistent with local stormwater management plans and with local floodplain management plans. If the Engineering District cannot obtain a written statement of consistency, the Engineering District must provide sufficient documentation with the permit application to demonstrate consistency with local plans implemented under the Acts.
4. For Joint Permit Applications, the Engineering District shall consider local land use plans.
5. The Engineering District shall complete and sign the appropriate permit application form or letter and send three sets of the permit application packet (including a completed application form or letter, and the required attachments) to the appropriate regulatory reviewing authority or submit the permit application to them through the JPA2 system. More or fewer sets of the permit application packet may be required by the regulatory review authorities, depending on the type of permits or nature of permit requests involved. For Joint Permit Applications, the primary regulatory review authority

will coordinate review of the application with other reviewing agencies when and as necessary (except for the U.S. Coast Guard, or as otherwise notified by the primary review authority on a case by case basis).

6. The Bureau of Project Delivery shall select at random three Hydrologic and Hydraulic Reports per district per year from the submitted reports (in hardcopy or through the JPA2 System) and will perform Q/A reviews and provide comments, if any, within fifteen calendar days on selected projects.
7. Where there are unusual or controversial special permit conditions specified on the permit by the regulatory authority, the Engineering District should consult with the Bureau of Project Delivery prior to acceptance of the permit.
8. The Engineering District shall submit one copy of each permit, or regulatory approval, (including permits or approvals received from the U. S. Army Corps of Engineers and the Pennsylvania Fish and Boat Commission) to the electronic document management system.

Additionally, please follow the document retention guidelines.

Please note that the Engineering District is responsible for processing and obtaining all necessary regulatory permits (such as the U. S. Army Corps of Engineer's Section 10 and U. S. Coast Guard Bridge Permits for proposed activities in navigable waters of the United States).

1.9.2.1 Hydrologic and Hydraulic Coordinators and Regulatory Permit Coordinators

Each Engineering District shall appoint a Hydrologic and Hydraulics (H&H) Coordinator and a Regulatory Permits Coordinator. These Coordinators are responsible for coordinating the processing of all hydrologic and hydraulic reports and all regulatory permits (or approvals) respectively. Jointly, these coordinators shall be knowledgeable in both administrative and technical aspects of the Hydrologic and Hydraulic Report and the permit application package. It is suggested that these coordinators be selected from the District's Bridge Unit and the District's Environmental Unit respectively.

1.9.2.2 Records of Regulatory Permits

The Engineering District shall maintain complete records of the Hydrologic and Hydraulic Reports and the permit documents sent to regulatory agencies for permit approval. Joint Permit Applications submitted to DEP through the JPA2 System inherently meet this requirement.

1.9.2.3 Permit Amendments

Regulatory permits for construction activities involving highway structures usually are issued at an early stage in the design process. During later stages of the design process, or during construction, issues may arise that cause changes in the information submitted to regulatory agencies. All changes in design after permits are issued must be weighed carefully against the regulatory permit review criteria (for example, see 25 Pa.Code §105.14, 151, and 161), and the information provided in the permit application. Early consultation and coordination with the regulatory agencies is encouraged. Minor design changes may be eligible for an amendment by letter. For large changes, the regulatory agency may process the amendment request in the same manner that it processes a new application. Primary responsibility for permit amendments during the design or construction phases of project development resides with the party responsible for design or construction respectively.

When design changes or construction changes are proposed that affect, or may affect, a regulatory permit, the following steps shall be followed:

1. The party responsible for any such changes shall carefully evaluate the effects of the proposed changes on the information provided in permit applications, issued permits, and conditions attached to issued permits. If any of these items are, or may be affected, then proceed to step 2.
2. The District's Regulatory Permit Coordinator will determine whether or not a permit amendment is necessary. Provide all information to the Regulatory Permit Coordinator necessary to enable a sound decision on whether or not an amendment is needed. The Regulatory Permit Coordinator will contact the regulatory agency/ies when necessary. If the Regulatory Permit Coordinator determines that a permit amendment is necessary, then proceed to step 3.

3. During coordination with the regulatory agencies, the Regulatory Permit Coordinator will determine whether or not the proposed amendment is minor. If the amendment is minor, then proceed to Subsection (a) below; otherwise, proceed to Subsection (b).
 - a. Minor Amendments
 - i. Compile all changes in the information submitted with the original permit application.
 - ii. Provide a certification with signature and professional seal for the changes as required by 25 Pa. Code §105.13(I).
 - iii. Provide a transmittal letter. The letter should be addressed to the regulatory review authority (i.e. DEP or a County Conservation District) from PennDOT. The letter shall provide a brief summary of the proposed changes; state that the changes are minor; and request a letter of amendment for a minor change.
 - iv. Submit a package of the above items to the attention of the Regulatory Permit Coordinator at PennDOT's District Office. PennDOT will review the materials.
 - v. If satisfactory, the package will be approved, signed, and forwarded to the regulatory review authority; otherwise, revision and resubmission will be necessary.
 - b. Other Amendments
 - i. For non-minor amendments, the approval process at the regulatory agency follows the same process as a new permit application. In the case of Individual permit applications and Small Project permit applications, this includes publication in the Pennsylvania Bulletin for public comment and publication of notice when the amendment is approved.
 - ii. Compile all changes in the information submitted with the original permit application. Follow the procedures described in PP1.9.2, items 1 through 8 above. Be sure to include new notices to local municipalities. Provide replacement pages or replacement sections for the permit application and its attachments for all parts of the permit application affected by the proposed changes.
4. The Engineering District will forward one copy of the amendment package to the Bureau of Project Delivery for Q/A review at the same time that it is sent to the regulatory review authority.
5. When a permit amendment is approved, the Engineering District will receive a Letter of Amendment from the regulatory review authority.
6. The Engineering District will scan the Amendment Letter to the electronic document management system.
7. The Letter of Amendment becomes a part of the original permit and must be permanently attached to the original permit.

1.9.2.4 Permits for Projects with Alternate Structures at One Location

For major structures (re: PP1.9.2, item 2.e), PennDOT may require regulatory permitting for more than one structure design at one location.

Attachments to the permit application should follow the outline recommended by the permit application forms. For individual items such as the Hydrologic and Hydraulic Report, the sections most affected when analyzing alternate structures at one location will be the hydraulic analysis, the risk assessment, and the summary data tables. With alternate structures, the report outline should not change; however, within each of these sections, each alternate should be analyzed independently. For example, the hydraulic analysis should contain subsections for Alternate 1, Alternate 2, etc. Other affected sections should be handled similarly.

1.9.2.5 Permits for Design-Build Projects

Regulatory permitting for structures does not depend on whether construction contracting uses the traditional procedures or the newer Design-Build procedures. Either the Department or the Design-Build Team will obtain the regulatory permits for Design-Build projects. Permits for Design-Build projects may be required to include alternate structures at one location as discussed in PP1.9.2.4 and PP1.11. Design changes subsequent to permitting must follow the process for permit amendments described in PP1.9.2.3.

1.9.3 Type, Size and Location (TS&L)

1.9.3.1 General

The investigation of a proposed structure shall be sufficiently intense to discriminatingly select and justify type, size and location on the basis of the information available from the various phases of study outlined in Design Manual, Part 1, including any foundation information obtained. Preliminary cost comparisons shall be made to support TS&L recommendations.

The District Bridge Engineer or his representative shall attend the Design Field View for all bridge projects to provide input in finalizing location, horizontal and vertical alignment for the project, taking into account site specific conditions, such as slide or scour potential and railroad clearances. Whenever an existing foundation is to be reused and new loads are to be applied, the existing foundations must be re-evaluated to assure adequate foundation carrying capacity. For bridge projects over Railroad facilities, early coordination and involvement with the Railroad shall be in accordance with the latest edition of Publication 371, Grade Crossing Manual, Chapter 4. This would allow for the Railroad's input to various design parameters being considered.

TS&L for any structure supported on proprietary walls shall not be approved unless adequate foundation information including scour evaluation (if applicable) is available or foundation investigation is completed and recommendations are available.

Type of substructure will be approved during foundation approval.

Formal TS&L approval is required for in-house designed BRADD projects.

Submit the TS&L Report and Structure Geotechnical Foundation Report for Retaining Walls and Sound Barrier Walls concurrently.

1.9.3.2 Responsibility

Refer to Tables PP1.9-1 and PP1.9-3 for the review and approval responsibility for TS&L. For new bridge designs having a deck joint at a substructure unit, the TS&L must be submitted to BDTD for approval.

1.9.3.2.1 Responsibility of District

The designer shall submit two sets of TS&L plans and related information (see PP1.9.3.3) to the pertinent District for approval when the District is responsible for TS&L approval.

The District shall send to the BDTD an informational copy of the final TS&L approval letter, with road plans, applicable Q/A forms and preliminary bridge plans showing core boring layout.

1.9.3.2.2 Responsibility of BDTD

The District Executive shall submit to BDTD, for approval, two sets of TS&L plans and related information (see PP1.9.3.3) for approval when BDTD is responsible for TS&L Approval.

If the District desires to revise a Consultant's submission, the revision shall be marked on the plans in red and forwarded to BDTD with an explanation where necessary.

BDTD will review the submission and will approve it if it is found satisfactory and after obtaining necessary FHWA approval. Submission of revised preliminary plans will be requested, if necessary.

1.9.3.3 Submission Requirement

TS&L Submission requirements are divided into two categories, Standard and Streamlined. Standard TS&L submissions are required on all projects unless a Streamlined TS&L submission is agreed upon by the District, BDTD and FHWA (if applicable).

1.9.3.3.1 Standard TS&L

The following information shall be included for TS&L submission:

- (a) TS&L submission letter

The letter of transmittal shall include the following:

- (1) Location - Over or under S. R. or local road, segment, offset, and station (and/or stream name, railroad name, or road name)

- (2) Type of superstructure recommended - Girder size and spacing and deck overhang dimensions

Indicate the type of superstructure coating system for steel girders (i.e. Painting, Galvanizing, Metallizing or None [for weathering steel bridges]).

- (3) Span C/C bearing and/or C/C piers
 - (4) Roadway width Out-to-out, curb-to-curb and sidewalk width where applicable
 - (5) Skew angle or range of skew angles and direction (left or right)
 - (6) Vertical and horizontal clearance: Minimum required, actual provided
 - (7) Type of substructure recommended
 - (8) Location, type and movement classification of proposed deck joints
 - (9) Bearing type and location (defer designation of bearing fixity in multi-pier structures until final design)
 - (10) Deck and off structure drainage
 - (11) Design methodology to be used for superstructure design
- (b) TS&L plans

The following information shall be shown on TS&L plans:

- (1) Plan view, including controlling clearances, span length, skew, existing contours and finished contours (excluding BRADD plans), scupper locations, and end structure drainage, where required
- (2) Elevation view showing controlling clearances, span length, existing and finished ground line, continuity, fix-expansion support condition, type and movement classification of expansion dams, and type of bearings
- (3) Cross-section showing out-to-out (O/O) dimension, traffic lanes, shoulder widths, beam type, size and spacing, overhangs, cross slope, superelevation, minimum slab thickness, type of traffic or pedestrian barrier, and thickness of wearing surfaces
- (4) Typical sections showing limits of individual construction stages where staging is required for construction of the bridge. Locations of longitudinal joints in the deck, locations and the type of temporary barrier, and traffic lane locations and widths shall be shown.
- (5) Elevation view of pier(s) showing proposed configuration
- (6) Deck protective system (for rehabilitation projects only)
- (7) Loading, design and analysis method; non-standard details
- (8) Core boring layout
- (9) Hydraulic information including design flood data, flood of record and date, slope protection, where required, and preliminary scour information
- (10) Horizontal and vertical curve data for all roadways shown
- (11) For retaining walls, the length and height for each segment (Note that the TS&L for walls will not be approved until foundation recommendation is provided.)

- (12) Bridge-mounted lighting poles, sound barriers and signs, if required.
- (c) Report on alternate studies and justification for the recommended bridge types
- (1) Cost comparison for all types considered during type, size and location study. (The cost estimate shall be arranged so as to indicate total cost per substructure unit and major portion of superstructure, e.g., rolled beam span, plate girder span.)
 - (2) Justification for recommended alternate
 - (3) Address the need to account for future widening and future redecking requirements into the recommended bridge
 - (4) Design Requirements for Contractor-Designed Alternate Structures and Design-Build Projects.
 - Permissible changes to the bridge geometrics (span, bridge width, abutments, and piers) and vertical and horizontal alignment.
 - Permissible Material Types (e.g., weathering steel, proprietary walls, etc.).
 - Permissible Number of Deck Joints (typically, this will be the number of deck expansion joints provided in the as-designed structure; however, this limitation should not be so restrictive that it eliminates the use of individual superstructure material types for the alternate).
 - Future Redecking Requirements (as applicable)
 - Maximum Number of Permissible Construction Stages.
 - Number of Required Lanes.
 - Minimum Lane Width(s).
 - Lane Location Limitations (if any).
 - Need to Maintain Pedestrian Traffic.
 - Minimum Number of Beams.
 - Design requirements for the individual stages.
 - Future Widening Requirements (as applicable).
 - Environmental Requirements Related to the Structure (as specified in the environmental clearance document - EIS, EA, CEE, or EER).
 - Other.
- (d) Foundation exploration plans
- Submission requirements are discussed in PP6.3.1.
- (e) Additional information to be supplied by the designer
- (1) Route and section number, index map and segment/offset of limits
 - (2) Program under which project will be financed (Federal-aid classifications, 100% State-funded, Department Force, or special program) , the WBS code and MPMS number.
 - (3) Name of designer (Consultant or District Office)
 - (4) List of proposed structures by station and type
 - (5) Design traffic data including current and projected ADTT and class of highways on relevant roads
 - (6) Date of line and grade approval and design speed
 - (7) Statement on balance of earthwork for project

- (8) Statement whether project is designed for free or controlled access
- (9) Prints or roadway plans showing approved typical sections; also pavement-type approval when available
- (10) Copy of waterway approval (from Department of Environmental Protection) and results of acidity tests of water and soil, if applicable
- (11) Copy of the minutes of the Design Field View approval as defined in Design Manual, Part 1, Chapter 2, and available road plans.
- (12) For rehabilitation projects, the following information shall be provided:
 - a. Age of existing structure, present and cumulative ADTT, portion to be replaced, type of steel-for-steel bridges, date of last inspection, type of diaphragm connections, i.e., welded or riveted, type and location of deterioration, deck drainage, expansion dam type, barrier type, and other pertinent items.
 - b. Live load ratings of the bridge at present and after rehabilitation.
 - c. Fatigue-prone details, such as out-of-plane bending problem areas, cover-plated beams, remaining fatigue life with and without retrofit, fatigue problems observed during inspection, recommended retrofit for existing fatigue-prone details, and other pertinent items.
 - d. Proposed scope of work.
- (13) For structures involving railroads, the following information shall be provided:
 - a. Completed Form D-4279 "Railroad Crossing Data for Design", as well as railroad right-of-way cross-sections, 500 ft. each side of proposed structure, degree of track curvature and rate of superelevation, if applicable.
 - b. Existing railroad drainage facilities and conditions in the vicinity of the structure site shall be investigated and described.
 - c. For situations in which railroads are overpassed by a highway structure, the procedures to determine track clearances are discussed in Design Manual, Part 1C, Chapter 4, Section 11D "Clearance of Track where Railroads are Overpassed by a Highway Structure", D2.3.3.4, and Publication 371, Grade Crossing Manual.
 - d. All contacts with the railroad companies shall be through the District Executive unless authorization is given to consultants, in which case copies of all correspondence and memoranda of meetings shall accompany submission of plans to the District.
 - e. A copy of the railroad company's letter of approval of acceptance regarding horizontal and vertical clearances, type of design live loading, type of steel and allowable stresses for various structural members shall be submitted with TS&L submission, as well as a request for temporary support for railroad tracks, if needed.
 - f. Demolition procedures including a schematic plan shall be provided for the removal of structures over or adjacent to railroads. The procedures and schematic must be coordinated with railroad representatives (see item d above).
- (14) Copies of all available structure foundation exploration from Design Manual, Part 1C, Chapter 4, Section 11F and PP6. (Provide a statement concerning mining in the area and any previous foundation problems, if any.)
- (15) Pedestrian count and information concerning possible future development which might warrant need for sidewalks and/or pedestrian protective fence.

- (16) Address problem areas so that there are no surprises at the final plan submission (kink in girders rather than curved girders, etc.). If problems or questions arise after approval is given, they should be brought to the attention of the Department.
- (17) Address safety areas which are structure related and were noted at the Design Field View.
- (f) Completed applicable Q/A Forms D-501, D-502, D-503 and/or D-504 (refer to Appendix A).

1.9.3.3.2 Streamlined TS&L

A Streamlined Submission, as a result of a Bridge Pro-Team meeting, shall include the information outlined in PP1.9.3.3.1(a) with signature blocks, (b) and (f).

The submission shall also include meeting minutes from the Pro-Team Bridge Scoping capturing all alternates discussed with reasoning behind decisions to pursue or exclude. All involved parties including the appropriate reviewing authorities, not just meeting attendees, should review and approve the minutes. The minutes should be made available to consultants for review during agreement advertisement for projects where consultants are used for design.

1.9.4 Foundations

1.9.4.1 General

The foundation exploration and report preparation shall be done as outlined in Chapter 6, and outlined herein.

Submit the TS&L Report and Structure Geotechnical Foundation Report for Retaining Walls and Sound Barrier Walls concurrently.

1.9.4.2 Responsibility

Refer to Table PP1.9-2 for the review and approval responsibility for foundations.

1.9.4.2.1 Responsibility of District

The foundation approval may be granted by the District Bridge Engineer or designee: However, input from the District Geotechnical Engineer should be considered. The District may consult BDTD and the Geotechnical Engineers and Geotechnical Section of the Innovation and Support Services Division about unusual cases.

The designer shall submit two sets of foundation plans to the District for approval.

1.9.4.2.2 Responsibility of BDTD

The designer shall submit to the District three sets of foundation plans for PennDOT oversight projects and four sets for Federal oversight projects.

The District Bridge Engineer and the District Geotechnical Engineer shall review the submission. The District Executive shall forward the submission to BDTD with the District's recommendation. The District shall submit to BDTD one set of the foundation submission for BDTD approval responsibility and two sets for FHWA approval responsibility projects.

If the District desires to revise the Consultant's recommendations, the revision shall be marked on the plans in red, with an explanation where necessary.

BDTD will review the submission and, after obtaining necessary FHWA approval, will approve it if it is found satisfactory. Submission of revised plans will be requested if necessary. The Geotechnical Section of the Innovation and Support Services Division may be consulted about unusual or complex foundations.

1.9.4.3 Submission Requirement

Foundation Submission requirements are divided into two categories, Standard and Streamlined. Standard Foundation submissions are required on all projects unless a Streamlined Submission is agreed upon by the District, BDTD and FHWA (if applicable).

1.9.4.3.1 Standard Foundation

The following information shall be included in the foundation submission:

(a) Foundation submission letter

The letter shall include the following for each substructure unit:

- (1) Proposed bottom of footing elevation.
- (2) Applicable core borings - B1, B2, etc. Identify bearing stratum. Show percentage of gross recovery and RQD for rock stratum and reasons for low gross recovery and/or RQD when applicable. For spread footing on soils, show average N value below the footing elevation to a depth equal to 1.5 times the width of the footing. For footings on piles, show average N value at least 10 ft below the estimated pile tip.
- (3) Rock and/or soil data for each layer below the footing, listed on Figures 1.9.4.3-1 and 1.9.4.3-2, used to compute the nominal foundation bearing resistance in tsf. (See also QA Form D505)
- (4) Pile type and size, pile tip reinforcement when required, rock and/or soil data for each layer below the footing, listed on Figure 1.9.4.3-3, used to compute the pile load resistance and driving method.
- (5) Estimated pile tip elevation, bottom of pedestal, drilled shaft and length of socket into bedrock for each substructure unit when applicable.
- (6) Caisson or Micropile type, size and the rock and/or soil data for each layer below footing, listed on Figure 1.9.4.3-3, used to compute the resistance.
- (7) Estimated settlement for footings on soils, fill material settlement that may affect the foundation, etc.
- (8) Scour depth for each substructure unit, if stream crossing.
- (9) If piles, caissons or micropiles are in a corrosive environment, submit information as stated in D10.7.5.5P.

(b) Foundation plans

The following information shall be shown on foundation plans:

- (1) Preliminary plans (dated), including plans and elevation showing type and elevation of the bottom of the footing and elevation of pile tip, recommended for each substructure unit.
- (2) Soil profile along the substructure units and longitudinal profile along the centerline of the structure (for uneven bearing stratum or when requested by the engineer).
- (3) Plotted logs of core borings and boring layouts, grouped for each substructure.
- (4) If settlement is a problem, a settlement control scheme or mitigation plan, after approval, shall be included in the final plans or in the proposal.
- (5) Foundation investigation information: The subsurface exploration data which are used in making recommendations concerning foundations shall include an *endorsement*, including the date and signature of a qualified geotechnical engineer or engineering geologist, stating that the information, as submitted, accurately represents the conditions encountered by the test boring program, including boring logs, earth samples, rock cores, classification of materials, and depth of borings.

Furthermore, the boring log sheets that are part of the bridge plans shall have the following note and initials of the geotechnical engineer or engineering geologist on each sheet: "The classifications of the materials encountered have been verified." (Initials).

For bridge construction plans prepared by District personnel, the District Geotechnical Engineer shall verify the accuracy of foundation data secured by drilling contractors or Department forces.

Endorsement shall be shown on the first sheet of the plotted test borings. In addition, the following note shall be shown on each test boring sheet: "This sheet is included for the convenience of the Department. Refer to Publication 408 Section 102.05 for further information."

The plotted test borings shall include all information contained on the boring logs.

- (6) Pile type, size and tip reinforcement for pile-supported footings. (See also QA Form D505)
- (7) Nominal size of drilled shafts, including the rock socket for footings supported on caissons.
- (8) Identification of substructure unit at each test boring, elevation of bottom of footings at each test boring, and elevation of bottom of pedestals and/or drilled shafts and/or pile tip.
- (9) Finished ground elevation at face of abutments or piers along the roadway, stream, or railroad.
- (10) Approved "Contour Grading and Drainage Plan" for interchange areas and other areas when applicable.
- (11) For foundations of structures over or along a stream, scour computation for abutments, piers and retaining walls and proposed scour countermeasures (including size and extent of riprap) calculated using finalized hydraulic data shall be included in the report.
- (12) Foundation information of existing or nearby structures (type of foundation, footings elevations, sign of settlement due to scour, etc.).
- (13) For bridge replacement structures over a stream or river, submit the following information on the *existing* structure:
 - a. Date built
 - b. Type of superstructure
 - c. Type of substructures
 - d. Type of foundation and piles if known and applicable
 - e. Bottom of footing elevation
 - f. Stream bed elevation
 - g. Waterway opening larger or smaller than proposed structure
 - h. Any scour or settlement due to scour
 - i. Debris accumulation problem
 - j. Containment within banks or lack of containment of upstream flood water
 - k. Reason for structure being replaced
 - l. Reason for substructure failure, if applicable
 - m. Approximate value of bearing pressure or pile load

For additional information concerning release of information and documents, see PP1.6.3.4.

(c) Geotechnical Report

In addition to the requirements specified in PP6.3.4.8, include foundation alternates studied (spread footings, piles, caissons, pedestals, etc.) including possible use of prefabricated proprietary walls, cost comparison when applicable, available driller's and Engineer's logs, and justification for the recommended foundation type, including allowable settlement or ultimate strength. If settlement is a problem, a settlement control scheme or mitigation plan, after approval, shall be included in the final plans or in the proposal.

(d) Completed Q/A Form D-505 for Foundations (refer to Appendix A).

SOIL BEARING RESISTANCE - THEORETICAL, SPT OR CPT METHODS FOR SPREAD FOOTINGS

			ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
Soil Type:			_____	_____	_____	_____	_____
% Gross Recovery:			_____	_____	_____	_____	_____
Average N Value:			_____	_____	_____	_____	_____
Unconfined Comp. Test Result (tsf):			_____	_____	_____	_____	_____
Coeff. of Friction between soil & footing:			_____	_____	_____	_____	_____
Soil Density:			_____	_____	_____	_____	_____
D_f (ft):			_____	_____	_____	_____	_____
B (ft):			_____	_____	_____	_____	_____
Show the appropriate information for A, B or C	(A) Theoretical Estimation Method:	c :	_____	_____	_____	_____	_____
		ϕ_{fm} :	_____	_____	_____	_____	_____
		N_γ :	_____	_____	_____	_____	_____
		N_c :	_____	_____	_____	_____	_____
		N_q :	_____	_____	_____	_____	_____
	(B) SPT Method:	$\bar{N}_{160}$:	_____	_____	_____	_____	_____
	(C) CPT Method:	q_c :	_____	_____	_____	_____	_____
q_{ult}^* -STRN (tsf):			_____	_____	_____	_____	_____
Sliding Resistance Factor, ϕ_τ :			_____	_____	_____	_____	_____
Bearing Resistance Factor, ϕ_b :			_____	_____	_____	_____	_____
q_o - SERV at anticipated settlement (tsf):			_____	_____	_____	_____	_____
Estimated Total Settlement (in):			_____	_____	_____	_____	_____
Estimated Settlement After Beam Erection (in):			_____	_____	_____	_____	_____
Tolerable Net Settlement (in):			_____	_____	_____	_____	_____

* q_{ult} is based on the assumed parameters as appropriate for cohesive or non-cohesive soils:

For all Methods:

$B/L = 1.0$ $(H/V)_{unfactored} = 0.25$
 ground water at $1.50 B + D_f$ below surface
 Footing in General Shear Failure Mode

These approved parameters will be used to calculate q_{ult} by the Department's Substructure Programs using the appropriate modification factors and load combinations.

Figure 1.9.4.3-1 – Rock Bearing Resistance Form

ROCK BEARING RESISTANCE - SEMI-EMPIRICAL OR ANALYTIC FOR SPREAD FOOTINGS

				ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
Bearing Stratum:				_____	_____	_____	_____	_____
RQD:				_____	_____	_____	_____	_____
Coeff. of Friction between rock & footing:				_____	_____	_____	_____	_____
D_f (ft):				_____	_____	_____	_____	_____
B (ft):				_____	_____	_____	_____	_____
Show the appropriate information for A or B	(A) Semi-Empirical Method	Use a or b	(a) RMR:	_____	_____	_____	_____	_____
			(b) NGI:	_____	_____	_____	_____	_____
			C_o :	_____	_____	_____	_____	_____
			N_m :	_____	_____	_____	_____	_____
				_____	_____	_____	_____	_____
	(B) Analytic Method	Closely Spaced Joints (D10.6.3.2.3aP)	c:	_____	_____	_____	_____	_____
			ϕ_{fm} :	_____	_____	_____	_____	_____
			Joint Spacing, S:	_____	_____	_____	_____	_____
			Open Joints K_e :	_____	_____	_____	_____	_____
			Closed Joints N_c :	_____	_____	_____	_____	_____
		Widely Spaced Joints (D10.6.3.2.3bP)	N_q :	_____	_____	_____	_____	_____
			N_γ :	_____	_____	_____	_____	_____
			γ :	_____	_____	_____	_____	_____
			J:	_____	_____	_____	_____	_____
			N_{cr} :	_____	_____	_____	_____	_____
	H_p :	_____	_____	_____	_____	_____		
	K_e :	_____	_____	_____	_____	_____		
q_{ult} -STRN (tsf):				_____	_____	_____	_____	
Sliding Resistance Factor ϕ_τ :				_____	_____	_____	_____	
Bearing Resistance Factor ϕ_b :				_____	_____	_____	_____	
q_o – SERV at anticipated settlement (tsf):				_____	_____	_____	_____	
Estimated Total Settlement (in):				_____	_____	_____	_____	
Tolerable Settlement (in):				_____	_____	_____	_____	

For Rocks defined as very poor quality in the Semi-Empirical Method use Soil Bearing Form with an equivalent soil mass.

These approved parameters will be used to calculate q_{ult} by the Department's Substructure Programs

Figure 1.9.4.3-2 – Rock Bearing Resistance Form

PILE RESISTANCE - SEMI-EMPIRICAL, SPT OR CPT METHODS FOR PILES IN SOIL

				ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
Soil Type:				_____	_____	_____	_____	_____
Static Analysis Method:				_____	_____	_____	_____	_____
Pile Size (e.g., HP 12 x 74) or Pile/Shaft Diameter (ft.):				_____	_____	_____	_____	_____
Micropile drill hole diameter through bonded length (ft.):				_____	_____	_____	_____	_____
Pile/Shaft Length (ft.):				_____	_____	_____	_____	_____
Rock Socket Depth/Micropile bonded length (ft.):				_____	_____	_____	_____	_____
Undrained Shear Strength, S_u :				_____	_____	_____	_____	_____
Show the appropriate information for A, B or C for friction and end bearing piles and caissons	(A) Semi-empirical Method:	Show the appropriate information for a, b, c or d	(a) Nordlund					
			K_δ :	_____	_____	_____	_____	_____
			P_d :	_____	_____	_____	_____	_____
			C_F :	_____	_____	_____	_____	_____
			$\delta:\delta$:	_____	_____	_____	_____	_____
			(b) α :	_____	_____	_____	_____	_____
			(c) β :	_____	_____	_____	_____	_____
			σ'_v :	_____	_____	_____	_____	_____
			λ :	_____	_____	_____	_____	_____
			(d) σ'_v :	_____	_____	_____	_____	_____
	(B) SPT Method:		N_{160} :	_____	_____	_____	_____	_____
			$\bar{N}_{160}$:	_____	_____	_____	_____	_____
			D_b :	_____	_____	_____	_____	_____
			D :	_____	_____	_____	_____	_____
σ'_v (tsf):			_____	_____	_____	_____	_____	
(C) CPT Method:		q_{c1} :	_____	_____	_____	_____	_____	
		q_{c2} :	_____	_____	_____	_____	_____	
(Requires approval of Chief Bridge Engineer)		Q_s :	_____	_____	_____	_____	_____	
			_____	_____	_____	_____	_____	

Figure 1.9.4.3-3 – Pile Resistance Form

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
Shaft Resistance (kips):	_____	_____	_____	_____	_____
Tip Resistance (kips):	_____	_____	_____	_____	_____
Total Resistance (kips):	_____	_____	_____	_____	_____
ϕ^* :	_____	_____	_____	_____	_____
Group Resistance:	_____	_____	_____	_____	_____
Is pile buckling a consideration?:	_____	_____	_____	_____	_____
Pile Spacing (ft):	_____	_____	_____	_____	_____
Maximum Factored Vertical Load per Pile (kips):	_____	_____	_____	_____	_____
Unfactored Vertical Load per Pile (kips):	_____	_____	_____	_____	_____
Maximum Factored Lateral Load per Pile (kips):	_____	_____	_____	_____	_____
Unfactored Lateral Load per Pile (kips):	_____	_____	_____	_____	_____
Estimated Total Settlement (in):	_____	_____	_____	_____	_____
Estimated Lateral Settlement (in):	_____	_____	_____	_____	_____
Estimated Settlement Before Beam Erection (in):	_____	_____	_____	_____	_____
Tolerable Settlement (in):	_____	_____	_____	_____	_____

* ϕ may be increased to a value between that shown in Table D10.5.5.2.3-1 for the method used to calculate total resistance and the value shown for "load tests" based on the data in DM-4, Appendix F, if it applies to the site conditions under consideration.

Figure 1.9.4.3-3 – Pile Resistance Form (continued)

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3
Pile Length (ft):	_____	_____	_____	_____	_____
Bearing Stratum:	_____	_____	_____	_____	_____
Boring Number:	_____	_____	_____	_____	_____
Bottom of Footing Elevation (ft):	_____	_____	_____	_____	_____
Pile Type & Batter:	_____	_____	_____	_____	_____
Estimated Pile Length (ft):	_____	_____	_____	_____	_____
Estimated Pile Tip Elevation (ft):	_____	_____	_____	_____	_____
Strength Resistance Factor ϕ :	_____	_____	_____	_____	_____
Tip resistance (kips):	_____	_____	_____	_____	_____
Is pile buckling a consideration?:	_____	_____	_____	_____	_____
Pile Spacing (greater than 3*D?) (ft):	_____	_____	_____	_____	_____
Maximum Factored Vertical Load per Pile (kips):	_____	_____	_____	_____	_____
Unfactored Vertical Load per Pile (kips):	_____	_____	_____	_____	_____
Maximum Factored Lateral Load per Pile (kips):	_____	_____	_____	_____	_____
Unfactored Lateral Load per Pile (kips):	_____	_____	_____	_____	_____

Figure 1.9.4.3-4 – Point Bearing Pile Resistance Form

1.9.4.3.2 Streamlined Foundation

A Streamlined Submission, as a result of a Bridge Pro-Team meeting, shall include the information outlined in PP1.9.4.3.1(a), (b), (c) and (d). PP1.9.4.3.1(c) references additional requirements in PP6.3.4.8. PP6.3.4.8(a), (b), and (d)(2) need not be furnished if borings are provided. PP6.3.4.8(d)(5) may be omitted if discussed in the Pro-Team meeting and documented in Pro-Team minutes.

The submission shall also include a Foundation approval letter with signature blocks, to the District, listing items from PP1.9.4.3.1(a) and meeting minutes capturing all alternates discussed with reasoning behind decisions to pursue or exclude. All involved parties including the appropriate reviewing authorities, not just meeting attendees, should review and approve the minutes. The minutes should be made available to consultants for review during agreement advertisement for projects where consultants are used for design.

1.9.4.4 Special Considerations

Based upon past experience, the following list of precautionary items is provided:

- (a) Foundation in limestone/dolomite area shall be evaluated conservatively, i.e., use a smaller resistance per pile, provide grouting if necessary, etc. History of sinkhole activity must be checked.
- (b) Piles or other deep foundations shall be recommended for substructure units in flood plain unless the footing will be supported on bedrock. Exceptions must be evaluated with extreme caution.
- (c) Interference of inclined piles of the same and adjoining substructure units must be checked.
- (d) For structure widening, watch for undercutting of existing foundation. Foundation column alternates may be considered. Similarly, foundation adjoining operating railroad or other property must be evaluated for the use of foundations column, caissons, etc., to eliminate cost of sheet piling or other similar costly measures.
- (e) Foundations for non-flexible walls or substructure units must be set below the frost depth.
- (f) Pile overdrive requirements may be needed for Conemaugh (clay stone and clay shale), decomposed mica schist and similar formations, if load test history indicates such a need.

1.9.4.5 Foundation Approval

The following items shall be included, as a minimum, in the foundation approval letter:

- (a) All data outlined in PP1.9.4.3.1(a).
- (b) Reasons for lower than normal allowable foundation pressures, pile loads, etc.
- (c) Specific pile-driving method.
- (d) Precautionary notes (for example, "Note that piles will terminate on limestone bedrock and considerable variation in the pile tip elevations may result").
- (e) A note to the effect that a copy of the foundation approval letter is to be given to the field office for the Inspector's guidance during construction.

1.9.5 Final Review of Plans

1.9.5.1 Responsibility

Refer to Tables PP1.9-1 and PP1.9-3 for review and approval responsibility for final plans.

The review of the final plans shall be conducted by either the District or BDTD, whichever has the final plan approval responsibility.

1.9.5.2 Submissions

Two sets of prints of final plans, special provisions and Quality Assurance forms shall be submitted for review and approval to the District Executive for the bridge types for which the District is responsible. Three sets of prints (five sets for major, unusual, or complex bridges) of final plans, and special provisions and Quality Assurance Forms shall be submitted to the District for the bridge types for which the Chief Bridge Engineer has the responsibility for approval. The District shall submit two sets (four sets for major, unusual, or complex bridges) to BDTD for review and comments. If the District desires to revise the Consultant's submission, the revision shall be marked on the prints in red prior to being forwarded to BDTD. BDTD will send a set (three sets for major, unusual, or complex bridges) to FHWA for its review and comments when applicable.

For special projects, partial submissions, such as superstructure or substructure, may be made for early input and comments so that major items are resolved before plans are finalized.

The Quality Assurance forms submitted shall be all the required applicable forms in accordance with Appendix A. A copy of all calculations pages required by the form shall also be attached to each form. Additional calculations may be required upon request of the reviewing office.

1.9.5.2.1 Plan Review by Consulting Engineers

The District shall provide one set of review plans, special provisions, and design computations and the required applicable Quality Assurance forms in accordance with Appendix A to the review consulting engineer. Direct communication and correspondence between the design and review consultants shall be permitted, provided that copies of correspondence are forwarded to both the District and BDTD. Any deviation from standard design practices, design criteria and standards shall be approved by the Chief Bridge Engineer prior to its acceptance by the review engineer.

1.9.5.3 Checklist of Minimum Items

In addition to the items included in PP1.6, the following list of minimum items is provided for uniformity and as a reminder:

- (a) All pertinent items included in TS&L and Foundation submissions. The sheet or sheets of plotted core borings shall be the last sheet or sheets in the set of structure plans.
- (b) Applicable general notes, quantities in the prescribed format (See Construction Items Catalog (Publication 7) and ECMS 'Master Items' for appropriate unit measures), including utility installation items and alternate bid items, table of deck elevations, etc. All bridges shall have alternate bid item unless prior approval for one design is secured at the TS&L stage.
- (c) All new bridges and new bridge superstructures (for rehabilitation projects) shall be bid lump sum. However, items below footings shall be bid on a unit price basis, except test piles, which will be a lump sum item. In addition, where quantities can be well defined, items may be bid lump sum. In case of conflict, the Chief Bridge Engineer shall be contacted.
- (d) All design computations shall be submitted at this stage, and shall be completely checked with an index.
- (e) All drawings shall be thoroughly checked for correctness and accuracy and shall be initialed by the designer and checker.
- (f) Bridge type, size, location and foundation details shall match approvals.
- (g) Foundation bearing pressures, axial and lateral pile/caisson loads, and the horizontal force for checking against sliding shall be shown for the controlling condition for each substructure unit. Indicate the controlling limit state, whether maximum values control, whether temporary or final conditions control, and the factored force effects (i.e., factored bearing pressure, factored pile axial load) and resistance values associated with the controlling conditions. A summary of soil/rock properties at each layer used for design shall also be shown, including, as applicable, undrained shear strength, mass unit density, saturated unit density, cohesion, effective friction angle, and empirical rock bearing capacity.

EXAMPLE:

Factored Pile Axial Load	=	_____	kips (Strength 1)*
Factored Pile Axial Resistance	=	_____	kips (Strength 1)*
Factored Pile Lateral Load	=	_____	kips (Strength 1)*
Factored Pile Lateral Resistance	=	_____	kips (Strength 1)*

*Show actual controlling limit state.

- (h) If a construction item is not a standard item covered by Publication 408, a special provision shall be prepared and submitted. Construction item terminology shall match the construction item catalog.
- (i) Utility occupancy data, transportability of prefabricated structure components, inclusion of special provisions for hauling permit and review of routes for accessibility shall be provided.
- (j) Moment and shear envelopes, section properties for composite designs, prestressed notes, details for live load continuity for prestressed beams including continuity diaphragms, additional deck steel in negative moment area, dowel details (keying of diaphragm to substructure not permitted), deck pouring sequence, camber diagrams, etc. shall be shown in drawings.
- (k) Bearing type and size shall be provided. The tolerance values used for the bearing pad design shall be shown on the construction plans. Provisions for future superstructure jacking shall be considered. The construction plans should clearly indicate where and when jacking is required, and provisions for jacking points must be included in the design and detailing of the superstructure and substructure. The jacking forces should also be specified.
- (l) When consecutively fixed piers are utilized in a design, instructions for jacking the required deflection into the piers for proper positioning of the bearings under the beams shall be shown on the drawings. A table of dimensions shall be included showing the relative displacements that each pier must be moved for each 10° F temperature variation from the mid-range of the anticipated temperature extremes. The jacking instructions and displacement table are required for continuous steel superstructures to ensure proper alignment of the bearings at erection. The same information is required for prestressed concrete superstructures made fully continuous. For prestressed concrete superstructures made continuous for superimposed dead loads and live load, the jacking instructions and displacement table are required for displacements exceeding 1/2" between the following temperature ranges 20° F to 58° F and 78° F to 110° F.

The theoretical fixed point on the bridge, based on the relative stiffness and heights of the piers that are fixed, shall also be shown on the drawings.

- (m) Details for expansion dams, manufacturing and installation, shall match the standards unless approved by the Chief Bridge Engineer.
- (n) Prestressed adjacent box beams: Bearing area should follow the deck cross slope; however, possibility of beam twisting should be watched by comparing seat cross slope at each end of the beam. Longitudinal slope should match the combination of grade and camber.

Prestressed spread box beams: Bearing area can be level transversely. Longitudinal slope should match the combination of grade and camber. Beveled sole plate shall be provided when longitudinal slope of the beam seat exceeds 4%, and the beam seat shall be level in the longitudinal direction. Special care shall be exercised when the bearings are parallel to a substructure unit with sharp skew; in such instances bevel shall be in two directions.

For both prestressed adjacent and spread box beams, D14.7.6.3 provides bearing area and sole plate requirements. For box beams having a transverse beam seat slope, s_t exceeding 5% and placed on neoprene bearing pads thicker than 3.5 in., provide a note on the design drawings requiring the contractor to provide temporary lateral support to the beam during construction until the end diaphragms are cast and the shear blocks or dowel bars are installed per D14.7.6.3.9d1.2P

If an exception has been given for the deck slab overhang, include a note on the plans per D9.7.1.5.1P.

- (o) Steel structures: Deck pouring sequence (identify lateral support for compression flange); fracture control plan; Charpy V-notch requirements; identification of tension flange zones; diaphragm connections to girders (fatigue-prone details [i.e. out-of-plane bending] shall not be used); end rotation on skewed bridges (compensate in expansion dam movement classification unless deck block-out detail is used for the dam); direction of deck placement (skewed placement) to eliminate corner uplift; camber diagram (including differential camber between fascia and interior girders), appropriate overhang notes (see D6.10.3.2.5.2P and D9.7.1.5.1P), note per D14.7.6.3.5 if a lift-off condition is expected when beam is initially set on bearing pad, weld joint symbols, etc.
- (p) Prefabricated walls: Typical foundation detail, conceptual drawing with all needed locations, dimensions and elevations, allowable foundation pressure with settlement control plan (if applicable), construction procedure (where required), barrier connection details, general notes, temporary shoring (where required), drainage details, abutment details (if applicable), concrete wall abutting details, and other site-specific requirements.
- (q) Wall design: Clarify the use of wet or dry soil condition(s) for wall design in the first submission. Also, clarify how the designer validated assumptions on the construction or contract plans.
- (r) Completed applicable Q/A Forms D-506 through D-518 (refer to Appendix A).
- (s) Demolition procedures including a schematic plan as approved by the railroad at TS&L submission shall be provided for the removal of structures over or adjacent to railroads.

1.9.6 Final Plans

1.9.6.1 Responsibility

Refer to Tables PP1.9-1 and PP1.9-3 for final plans.

1.9.6.1.1 Responsibility of District Bridge Engineer

The District may consult BDTD about unusual cases. When Federal funds are used in any phase of a project, PennDOT oversight project procedure shall be followed.

Bridge-mounted sound barrier plans shall be approved for structural adequacy only.

1.9.6.1.2 Responsibility of BDTD

The District Executive shall submit the plans, special provisions, one set of prints and the review prints with BDTD's comments to BDTD for approval of the bridge plans.

BDTD will review and approve the plans and Special Provisions after satisfactory resolution of all comments, and will send prints of the approved plans, if necessary, or when requested by the District for preparing PS&E submission.

Bridge-mounted noise barrier plans shall be approved for structural adequacy only.

1.9.6.2 Plan Presentation

See PP1.6, "Plan Presentation". If the plans are prepared by a consulting engineer, the first sheet shall be signed and stamped by a Professional Engineer registered in Pennsylvania.

For design review performed by consultants, see PP1.3.4, which provides additional requirements.

All comments from the review of the final plans (PP1.9.5) shall be addressed before the final plans are approved.

1.9.6.3 Signing of Bridge Plans

The first sheet shall be dated and signed by the Chief Bridge Engineer or the District Bridge Engineer, depending on whose office has signature authority. All other sheets, except core boring sheets, shall be dated only. The core boring sheets shall neither be signed nor dated, except as indicated in PP1.9.4.3.1(b)(5).

The following procedure shall be followed for the approval of structure plans for local projects:

1. Federally-Funded Local Projects:

The structure plans shall be processed using appropriate (PennDOT oversight or Federal oversight) procedures. The District Bridge Engineer should sign the first sheet of the structure plans and indicate the approval date on the remaining sheets.

2. State-Funded Local Projects:

The structure plans shall be processed as specified in Chapter 3 of Publication 9. The District Bridge Engineer should sign the first sheet of structure drawings "For Structural Adequacy Only" and indicate the approval date on the remaining sheets.

If separate structure plans are not prepared, all structure-related drawings shall be distinctly separated (preferably at the end) from other drawings, such as highway plans or traffic plans, etc. In such case, the District Bridge Engineer shall sign the sheet where the structure drawings begin and indicate the approval date on the remaining structure sheets.

1.9.7 PS&E Submission

1.9.7.1 General

See Design Manual, Part 1C, Chapter 4. The bridge plans must be signed before submitting PS&E submission to the Contract Management Section, Highway Delivery Division of the Bureau of Project Delivery.

For Bridge rehabilitation or replacement projects incorporate the following with the PS&E submission to Central Office:

Indicate the availability of the existing bridge plans on the Title Sheet of the contract plans, as shown:

Sample Title Sheet
Also Included

Interconnection Plans	<u>4</u> Sheets
Traffic Signal Plans	<u>21</u> Sheets
Traffic Control Plans	<u>5</u> Sheets
Pavement markings & Signing Plans	<u>9</u> Sheets
Structure Plans S-21004	<u>2</u> Sheets
Landscaping Plans	<u>10</u> Sheets
Existing Bridge Plans (Upon Request)	
S-xxxxx	<u> </u> Sheets
S-zzzzz	<u> </u> Sheets

Include one set of existing bridge plans (half-size), as well as plans of any interim work done on the existing structures. Include only those plans and/or shop drawings which have been used and are appropriate in preparing the proposed contract bridge plans.

Stamp sheet "For Reference Only" on each existing bridge plan submitted.

The existing plans (half-size) and the construction plans will be sold to the prospective bidders, upon request, by the Contract Sales Store.

1.9.7.2 Responsibility of District Bridge Engineer

The District Bridge Engineer is responsible for correct item numbers, descriptions and quantities shown on the bridge plans, and this information shall be cross checked by the District with roadway plans. The Bridge Quantity Summary shall be shown on the roadway plans.

1.9.7.3 Responsibility of BDTD

BDTD is responsible for reviewing the structure portion of the PS&E submission in accordance with Table PP1.9-1 and Table PP1.9-3 (where applicable).

1.9.8 Revisions to Contract Drawings and "As-Built" Plans

Refer to PP1.10.6.

1.10 BRIDGE SUBMISSIONS - CONSTRUCTION PHASE**1.10.1 Alternate Design by Contractors**

Submission of applicable Q/A forms per Appendix A is mandatory without which the submission will be returned without any action by the approving office. Any delay caused by such non-compliance will be the Contractor's responsibility.

1.10.1.1 General

Alternate bridge design by contractors shall be permitted for all bridge projects unless approval is secured for one bridge type at TS&L stage from the Director, Bureau of Project Delivery. Any constraint requirements shall be included in the "Alternate Bridge" special provision. See PP3.3.4 for submission requirements regarding prefabricated walls. Alternate bridge plans are considered a new set of design plans. Write the following note on the title sheet in the area of the structure (S) numbers:

"DRAWING S - ____ SUPERSEDES DRAWING S - ____"

Using the following policy for all alternate designs:

1. Significant changes to the "as-designed" structure, i.e., both superstructure and/or substructure redesign, requires the alternate designer to sign and seal the entire set of alternate drawings with no restrictions in responsibility.
2. In cases where only minor items are modified, i.e., change to specific substructure units, etc., the following note and seal combination is acceptable on the first sheet of alternate designs:

ALTERNATE DESIGN

This alternate encompasses a Redesign of (items redesigned) Sheets (affected sheets) and all related items

(Seal)

(Signature)

Prepared by:

After the project is completed, mylars of the alternate design developed by a Contractor and used to build the project shall be sent to the Bureau of Project Delivery, to the attention of Plans, Records and Reproduction unit of the Project Schedules, Specifications and Constructability Section of the Highway Delivery Division, for microfilming. An 8 ½" x 11" sheet is to be included showing county, S. R., section, station, S-number, letting date, and the statement: "These plans were used for bridge

construction in lieu of S____ and S____". This sheet will be microfilmed and placed on the front of the package. A different colored microfilm card will be used to identify Contractor's alternate.

When a contractor has used one of two designs supplied by the Department, only an 8 ½" x 11" sheet is to be included showing county, S. R., section, station, S-number, letting date, and the statement: "These plans were used for bridge construction in lieu of S____". This sheet shall be sent to the Bureau of Project Delivery for the design used.

1.10.1.2 Details for Alternate Design by Contractor

When the Contractor chooses to bid an alternate bridge based on his/her own design, the plans of the original design shall be provided by the District Office, after the project is awarded, upon request. After the conceptual approval is secured, the Contractor may change the original plans to reflect the alternate design, or may develop completely new drawings on reproducible linens or drafting film. The redesign plans will go through an approval process before actual construction begins, as explained in the bid proposal. The original design computations shall be loaned to the Contractor, upon request, when the plans of the original design are obtained. These computations shall be returned when the alternate design computations are submitted with the alternate design plans. The alternate design by the Contractor shall be submitted to the pertinent Engineering District Executive. Based upon the approval authority for the original design, the District Executive shall process the submission as shown in the following chart:

Chart for Review of Alternate Design Developed by Contractors

Original plans signed by Chief Bridge Engineer		Original plans signed by District Bridge Engineer	
PP1.10.1.2.1a	PP1.10.1.2.1b	PP1.10.1.2.2	PP1.10.1.2.2b
In-house review and approval	Consultant review and approval	In-house review and approval	Consultant review and approval

1.10.1.2.1 Federal Oversight Projects and Major, Unusual or Complex Projects

1.10.1.2.1a In-House Review and Approval

The following procedure shall be used:

- (a) The Contractor shall submit three sets (five sets for major, unusual and complex projects) of the conceptual design plans to the District Executive, of which two sets (four sets for major, unusual or complex projects) shall be sent to the Chief Bridge Engineer for approval, and one copy will be retained in the District Office. One set (three sets for major, unusual or complex projects) shall be sent to the FHWA for review and concurrence. The conceptual design shall include all basic details for the proposed bridge, plus the design methodology, including the type of computer program that will be used in the design, construction sequencing and any concepts or details not covered in design and construction specifications or standards, or practice not commonly used in Pennsylvania. A list of major items that deviate from the "as-designed" plans shall be attached to the submission. All applicable Q/A forms, D-501 through D-505, shall also be completed and submitted (refer to Appendix A).
- (b) The District Office shall submit general comments to the Chief Bridge Engineer to help in the decision on acceptance of the alternate. After the conceptual design is approved by the Chief Bridge Engineer, two sets (four sets for major, unusual, or complex projects) of detailed design plans and computations shall be submitted (when requested) to the Chief Bridge Engineer for review and approval. For major, unusual or complex projects, three sets shall be forwarded to FHWA for review and concurrence. Partial design plans and computations may be submitted for approval to expedite the project. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).

- (c) After each partial submission is approved, the Contractor shall submit 12 sets of the approved plans for distribution. These 12 sets shall be stamped "Recommended for Construction", signed by the Chief Bridge Engineer or his designee, and distributed as follows:
 - 1. six sets to Contractor
 - 2. three sets to District
 - 3. one set to consultant that developed the alternate design
 - 4. two sets for Bridge Division file, including one set for FHWA
- (d) Upon design completion, the Contractor shall submit the plans and computations to the Chief Bridge Engineer for signature, including all applicable Q/A forms completed and submitted under (a) and (b) above. The first sheet of the plans shall bear the name of the consultant that developed the alternate design and shall bear the professional seal and signature of the engineer responsible for the design.
- (e) Once the plans have been signed, one set of half-size prints shall be sent to the Contractor and one set (three sets for major, unusual and complex projects) shall be sent to FHWA. The Bridge Design and Technology Division will keep one set for the files. A set of microfilms will be made and kept by the Bureau of Project Delivery, Plans, Records and Reproductions unit of the Project Schedules, Specifications and Constructability Section of the Highway Delivery Division.
- (f) The plans and computations shall be sent to the pertinent District Office for additional copy distribution and holding until construction is completed.
- (g) After the project is completed and the plans are revised "as constructed", the District shall send the plans to the Bureau of Project Delivery for microfilming.
- (h) The Bureau of Project Delivery will send microfilms and plans back to the District for permanent storage.

1.10.1.2.1b Consultant Review and Approval

The following procedure shall be used:

- (a) The Contractor shall submit four sets (six sets for major, unusual and complex projects) of the conceptual design plans to the District Executive, of which two sets (four sets for major, unusual or complex projects) shall be sent to the Chief Bridge Engineer, one set to the review consultant, and one set will be retained in the District. The BDTD will send one set (three sets for major, unusual or complex projects) to FHWA for review and concurrence. See PP1.10.1.2.1a(a) for items to be included in the submission by the Contractor.
- (b) The Consultant will review and recommend the conceptual design to the Chief Bridge Engineer for approval. The District Office shall send general comments to the Chief Bridge Engineer to help in the final decision on alternate acceptance.
- (c) After the conceptual design has been approved by the Chief Bridge Engineer, the Contractor shall submit four sets (six sets for major, unusual or complex projects) of detailed plans and computations (if requested): one set to the Chief Bridge Engineer, one set to the District Office, one set to FHWA (three sets for major, unusual or complex projects) and one set to the review consultant for approval. Partial plans and computations may be submitted to expedite the project. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).
- (d) The review consultant will review and approve the partial submission. Upon approval by the consultant, the Contractor shall submit to the Chief Bridge Engineer 13 sets for Department acceptance and distribution. These 13 sets shall be stamped "Recommended for Construction", signed by the Chief Bridge Engineer, and distributed as follows:
 - 1. six sets to Contractor
 - 2. three sets to District
 - 3. one set to consultant that developed the alternate design
 - 4. one set to review consultant
 - 5. two sets for Bridge Division file, including one set for FHWA

- (e) After all partial submissions have been reviewed and approved, the Contractor shall submit the plans and computations to the review consultant for final verification.
- (f) The review consultant shall send the plans, after stamping them according to PP1.3.4, to the Chief Bridge Engineer with a letter indicating that the alternate design is satisfactory and recommending approval of the plans. The first sheet of the plans shall bear the name of the consultant who developed the alternate design and shall bear the professional seal and signature of the engineer responsible for the design.
- (g) The computations and all applicable Q/A forms completed and submitted under (a) and (c) above shall be sent directly to the District by the review consultant.
- (h) The Chief Bridge Engineer, upon approval of the plans, will send one set of half-size prints to the Contractor, one set (three sets for major, unusual or complex projects) to FHWA, and will keep one set for the office. A set of microfilms will be made and kept by the Bureau of Project Delivery, Plans, Records and Reproduction unit of the Project Schedules, Specifications and Constructability Section of the Highway Delivery Division. The plans will be sent to the District for distribution of additional copies and storage until the project is completed.
- (i) After the project is completed and the plans are revised to "as constructed", the District shall send the plans to the Bureau of Project Delivery for microfilming.
- (j) The Bureau of Project Delivery will send microfilms and plans back to the District for permanent storage.

1.10.1.2.2 PennDOT Oversight Projects

1.10.1.2.2a In-House Review and Approval

The following procedure shall be used:

- (a) The Contractor shall submit two sets of the conceptual design plans to the District Executive for approval. See PP1.10.1.2.1a(a) for items to be included in the submission by the Contractor.

If the Contractor's design employs a sophisticated design method or software, or includes superbeams or any unusual features or unusual foundation, the submission shall be sent to the Chief Bridge Engineer for approval. The approval procedure for Federal oversight projects shall be followed thereafter, unless directed otherwise by the Chief Bridge Engineer.

- (b) Upon approval of the conceptual design by the District Executive, one copy of the conceptual approval shall be sent to the BDTD for information and the file, and the Contractor shall submit two sets of prints of the final design and computations to the District for review and approval. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).
- (c) After the final plans have been approved, the Contractor shall submit the plans and computations to the District Executive for signature by the District Bridge Engineer. The first sheet of the plans shall bear the name of the consultant that developed the alternate design and shall bear the professional seal and signature of the engineer responsible for the design.
- (d) After the plans have been approved, the District shall distribute 11 sets of prints of the plans as follows:
 1. six sets to Contractor
 2. three sets for District use
 3. one set to consultant that developed the alternate design
 4. one set to BDTD for the file

The District, at this point, can make a set of copies for its use and send the plans to Bureau of Project Delivery for microfilming. After microfilming, the plans will be sent back to the District for revision during construction and for completion to "as constructed" plans. The Bureau of Project Delivery will hold the microfilms until "as constructed" plans are microfilmed as indicated below.

- (e) After the project is completed and the plans are revised "as constructed", the District shall send the plans to the Bureau of Project Delivery for microfilming.
- (f) The Bureau of Project Delivery will send the microfilms and plans back to the District for permanent storage.

1.10.1.2.2b Consultant Review and Approval

The following procedure shall be used:

- (a) The Contractor shall submit three sets of the conceptual design plans to the District Executive, of which one set shall be sent by the District to the review consultant. See PP1.10.1.2.1a(a) for items to be included in the submission by the Contractor.

If the Contractor's design employs a sophisticated design method or software, or includes superbeams or any unusual features or unusual foundations, the submission shall be sent to the Chief Bridge Engineer for approval. The approval procedure for Federal oversight projects shall be followed thereafter, unless otherwise directed by the Chief Bridge Engineer.

- (b) The Consultant shall review and, if acceptable, recommend the conceptual design for approval to the District Executive. One copy of the conceptual approval shall be sent to BDTD for information and the file.
- (c) After the conceptual design has been approved by the District Executive, the Contractor shall submit two sets of the final plans and design computations to the review consultant and one set to the District. Partial submissions may be made to expedite the review process. All applicable Q/A forms, D-506 through D-518, shall also be completed and submitted (refer to Appendix A).
- (d) The review consultant will review and approve partial submissions. Upon approval by the review consultant of the partial submission, the Contractor shall submit 12 sets to the District Executive for Department acceptance and distribution. These sets will be stamped "Recommended for Construction", signed by the District Bridge Engineer, and distributed as outlined in Step (g) below.
- (e) After all partial submissions have been reviewed and approved, the Contractor shall submit plans and computations to the review consultant for final verification.
- (f) The review consultant shall submit the plans, stamped according to PP1.3.4, and the original computations and all applicable Q/A forms completed and submitted under (a) and (c) above to the District Executive with a letter stating that the alternate design is satisfactory and that the review consultant recommends approval of the plans.
- (g) After the plans have been approved, the District shall distribute 12 sets of prints of the plans as follows:
 - 1. six sets to Contractor
 - 2. three sets for District use
 - 3. one set to consultant that developed the alternate design
 - 4. one set to review consultant
 - 5. one set to BDTD for the file
- (h) The District, at this point, can make a set of copies for its use and send the plans to the Bureau of Project Delivery for microfilming. After microfilming, the plans will be sent back to the District for revision during construction and for completing to "as constructed" plans. The Bureau of Project Delivery will hold the microfilms until "as constructed" plans are microfilmed as indicated below.
- (i) After the project is completed and plans are revised "as constructed", the District shall send the plans to the Bureau of Project Delivery for microfilming.
- (j) The Bureau of Project Delivery will send the microfilms and plans back to the District for storage.

1.10.1.3 Disposition of Unused Plans for Alternate Bridge Designs

While an unused alternate design is of no future use to the Department, it nevertheless represents evidence of a product received for design fees expended. It is the opinion of the Department's Chief Counsel that the microfilm of the unused alternate design plans is sufficient evidence of a product received for design fees expended. In addition, the microfilm of the unused design may be useful in future litigation.

Therefore, unused design plans shall be microfilmed and kept indefinitely. Design computations for unused designs need not be kept.

1.10.1.4 Changes in Prestressing by Contractor

The contractor is not permitted to modify the number, size or spacing of the beams or the design methodology without bidding an alternate design. For any other deviations from the original beam design, such as changes in the prestressing, the plan revision procedures found in PP1.10.6 are to be followed during the construction phase of the project.

1.10.2 Shop Drawings

Shop drawings submitted for acceptance shall be prepared by the Contractor in accordance with the requirements of the design drawings, Department standards, contract special provisions and Publication 408, Section 105.02(d). Shop drawings shall be properly reviewed and accepted before fabrication begins. The Department requires initial submissions to be electronic portable document format (PDF) files and requires final submissions for distribution to be prints with matching PDF files, as described in Publication 408, Section 105.02(d).

1.10.2.1 Shop Drawing Review

Technical guidelines for shop drawing review are given in Appendix B.

1.10.2.1.1 In-House Design

For all in-house designs developed by the Districts, the shop drawings shall be reviewed and approved by the District Bridge Engineer.

1.10.2.1.2 Consultant Design

The shop drawings for the Consultant's design shall be reviewed and approved as follows:

- (a) By the same consultant, if the shop drawing review item is included in the original engineering agreement or in a supplemental engineering agreement
- (b) By a different consultant using an open-end engineering agreement with the Districts or using a statewide open-end contract with Central Office
- (c) By the District Bridge Unit Staff

Consideration shall be given to retaining the original consultant for shop drawing review when the design is complex and particularly when fracture-critical members are involved.

1.10.2.2 Purpose

Shop drawing review is conducted to ensure that fabrication of items is in accordance with the intent of the contract, i.e., the design drawings, standards, specifications and special provisions.

1.10.2.3 Required Shop Drawings

Unless stated otherwise in the contract special provisions, the following items routinely require submission of shop drawings:

- (a) Fabricated structural steel including, but not limited to, the following:
 - (1) Primary and secondary members, such as girders, trusses, beams, framing systems, cross bracing, diaphragms and stringers
 - (2) Grid floors
 - (3) Expansion dams and fixed dams
 - (4) Railings and/or barrier, sidewalk or protective barrier
 - (5) Bearings (complex and simple bearing devices)
 - (6) Sign structures
- (b) Pre-tensioned, pre-post-tensioned and post-tensioned concrete beams and panels
- (c) Permanent metal deck forms
- (d) Metal plate culverts
- (e) Precast concrete culverts
- (f) Precast channel beams
- (g) Precast deck sections - Pretensioned, post-tensioned, or reinforced concrete
- (h) Precast bridge barriers and curbs (only if permitted by the contract)
- (i) Timber bridges
- (j) Impact attenuators
- (k) Reinforced concrete cribbing
- (l) Proprietary retaining walls (reinforced earth, retained earth, doublewall, etc.)
- (m) Anchored pile walls
- (n) Stud details
- (o) Sound barriers
- (p) Light poles
- (q) Protective Fence

1.10.2.4 Statewide Standardization and Quality Control

The Bureau of Project Delivery, Bridge Design and Technology Division, is responsible for developing and maintaining guidelines for the review and approval of shop drawings. Unique features or deviations from standard practice should be brought to the attention of BDTD, preferably through the District Bridge Engineer. When deviations from standard practices become repetitive, BDTD will distribute acceptable deviations to the Districts for uniformity. The use of District standards is discouraged.

1.10.2.5 Review Procedure

The General Contractor has the responsibility to inform the Subcontractors of the name of the Department's agent for shop drawing review. Prior to commencing review, the Reviewers shall ascertain that the contract has been awarded. Furthermore, it is important that the Reviewers are in possession of all the latest contract documents, i.e., design drawings, special provisions and supplements to Publication 408. The District is responsible for furnishing the Reviewers with all pertinent contract documents. The Reviewers must also have in their possession general reference material, such as Design Manual, Part 4, Bulletin 15, and Standard Drawings.

Generally, shop drawings shall be submitted for each structure individually (items pertaining to the same S-number). This procedure will facilitate bookkeeping and avoid confusion when microfilm records are made. Each drawing must contain a title block in the lower right-hand corner indicating the county, route, section number, segment and offset, station, contract number, name of Contractor, name of Fabricator, title of drawing, sheet number, design structure number (S-number), initials of the drawer, initials of the checker, and date of the drawing.

The Contractor is responsible for furnishing shop drawings to the Department in accordance with Publication 408, Section 105.02(d). Unless the Contractor insists on being the liaison through which the shop drawings flow from the Subcontractors to the Reviewer, the respective Fabricators may submit their shop drawings directly to the reviewing engineer contracted to do the shop drawing review.

The Contractor shall be kept informed about the progress of the review by copies of transmittal letters between the Reviewer and the respective Fabricators.

The following procedure shall be used:

- (a) Review the initial submission against design plans, contract with revisions, and all addenda up to the date of review. It is recommended that a print of each electronic drawing be produced and used for review, instead of reviewing the drawings on computer screen only, to improve cross-checking. When the review is complete add comments, corrections and status label directly to PDF file (either using writer software or using reader software if the commenting and signature features were enabled by the author), or scan marked-up print to produce new PDF file. Return files to Fabricator on compact disc or upload to the Fabricator supplied FTP web site (if available). As an alternate, files may be returned by e-mail provided attachments total to no more than 9.0 MB per e-mail message. Provide electronic transmittal forms regardless of return method. Provide a status label on each drawing in one of the following forms:
 1. "Initial Submission - Accepted" or
 2. "Initial Submission - Accepted as Noted" or
 3. "Initial Submission - Returned for Correction"
- (b) Files returned to the Fabricator marked either "Accepted" or "Accepted as Noted" shall be corrected by the Fabricator as required, and prints thereof resubmitted for final distribution. Files returned to the Fabricator marked "Returned for Correction" shall be corrected by the Fabricator and then resubmitted to the Reviewer in electronic form for further review. Continue procedure until files are returned marked either "Accepted" or "Accepted as Noted" and then prints thereof shall be resubmitted for final distribution after corrections, if any, have been made.
- (c) Review of prints submitted for final distribution should be conducted against design plans, contract with revisions, and all addenda up to the date of review. When the review is complete, transfer any comments and corrections to the remaining sets of prints and stamp the drawings in one of the following ways:
 1. "Accepted" or
 2. "Accepted as Noted" or
 3. "Returned for Correction"

Only after all drawings are stamped either "Accepted" or "Accepted as Noted" are they to be distributed in accordance with Figure 1.10.2.5-1. If any of the drawings are stamped "Returned for Correction", notify the Fabricator of the situation and arrange to have sufficient sets of replacement prints submitted and reviewed until all drawings are found satisfactory. Then distribute the drawings in accordance with Figure 1.10.2.5-1. If an ftp site folder is available, all shop drawings of a submission are to be combined into a single PDF file which is then uploaded.

- (d) Distribution of accepted shop drawings shall be in accordance with Figure 1.10.2.5-1.

SHOP DRAWING DISTRIBUTION					
ITEM	FABRICATIONS	CONTRACTORS	STRUCTURAL MATERIALS SECTION	PRESTRESSED CONCRETE FABRICATORS	DISTRICT EXECUTIVE
Metal Deck Forms	As many sets as desired	1 set*	---	---	3 sets ▲
All Other Shop Drawings	As many sets as desired	1 set*	2 sets**	---	3 sets ▲
▲ 1 - Str. Control Engineer 1 - District Bridge Engineer 1 - Construction Project Engineer *If a Contractor desires to have additional sets, may require the Fabricator to supply the additional sets, either directly or via the Department **1 set is to be sent directly to the Shop Inspector					

Figure 1.10.2.5-1 – Shop Drawings Distribution

- (e) When there is no shop inspection, as is the case for metal plate culverts or precast concrete culverts, small rehabilitation and repair jobs, etc., copies of correspondence and/or distribution of shop drawings shall be adjusted accordingly.
- (f) When a drawing is stamped accepted the only other note the reviewer may place on the drawings is as follows:

This shop drawing has been reviewed for the compliance with the design concept and strict adherence to all material, technical requirements and details indicated in the contract drawings, standards, and specifications. Approval does not relieve Contractor from their responsibility for design (if applicable), detail, workmanship, dimensions, and full conformance to the contract documents.

The following additional requirements shall be met:

- (a) The Fabricator shall deal directly with the Reviewer (unless the Contractor wants to be the liaison through which the work flows).
- (b) For outside review, the Fabricator and Reviewer shall furnish copies of pertinent correspondence and transmittal letters to the District for monitoring purposes.
- (c) All distribution prints of accepted shop drawings shall be affixed with the Reviewers acceptance stamp and distributed as required according to Figure 1.10.2.5-1.
- (d) The District shall keep a log of the activities related to review and distribution of shop drawings.

1.10.2.6 Permanent Records of Shop Drawings

Permanent records are required of the following shop drawings:

- (a) Structural steel
- (b) Grid floors
- (c) Expansion dams

- (d) Sign structures
- (e) Pretensioned and/or post-tensioned concrete beams
- (f) Timber bridges
- (g) Sound Barrier Walls

Shop drawing files shall be maintained until the job is paid off, or as the case may be, a prospective claim has been settled.

“As-built” shop drawings or microfilms thereof shall be furnished by the Contractor in accordance with Publication 408, Section 105.02(d), and shall be in the Department's possession before the job is paid off.

1.10.3 Pile Hammer Approvals

Districts shall submit to BDTD, for approval, the hammer operating specifications and all other additional data specified in Publication 408. Pile hammers may not be used without a formal pile hammer approval letter from the Director, Bureau of Project Delivery, specifying the approved pile driving parameters. The District Structural Control Engineer or the District Bridge Engineer is responsible for approval of the steel pile tip reinforcement attachment details and splice details based upon Standard Drawing BC-757M. This standard should be included as a supplement drawing for design projects where piles are used. If a submission for pile tip reinforcement attachment details and splice details are outside the scope of the Standard Drawing BC-757M, the approval should be sent to the Chief Bridge Engineer for approval.

1.10.4 Pile Load Test Evaluations

When a pile load test is conducted, continuous liaison shall be established between the field and BDTD, at which time the test results will be evaluated.

All pertinent test results shall be forwarded to BDTD as specified in the proposal.

1.10.5 Construction Problems

Design-related bridge construction problems shall be brought to the attention of the District Bridge Engineer. The District Geotechnical Engineer shall also be notified if the problem involves the geotechnical area of the bridge. All problems of structural integrity related to fabrication, erection, or construction items shall also be brought to the attention of the District Bridge Engineer.

The District Bridge Engineer shall approve modifications or corrections including change in foundation type, according to the approval authority of the original design. For all Federal oversight projects and major, complex or unusual structures, BDTD shall be contacted with detailed information for approval. BDTD will secure FHWA approval prior to approving the modifications.

If design error is suspected, the procedure outlined in Publication 93, Chapter 5, Section 5.7, "Design Error Process", shall be followed.

If a Contractor must perform a reanalysis of a design due to deficient strength concrete, then the required applicable Quality Assurance Form D-518 (refer to Appendix A) must be submitted along with their analysis.

1.10.6 Revisions to Contract Drawings and "As-Built" Plans

Once the contract drawings are signed and sealed by the designer-of-record, no changes are permitted unless the plan revision procedures described below are followed.

1.10.6.1 Revisions During Bidding Phase

The District is to revise the plans to reflect changes identified in addenda during the bidding stage. Issue the necessary copies of the revised plans to the successful contractor and all affected parties.

1.10.6.2 Revisions During Construction Phase

1.10.6.2.1 Revisions Due to Errors/Omissions or Field Conditions

The District will revise the contract drawings, as necessary, to correct errors or omissions uncovered during construction or to document changes necessitated by field conditions. The District will make the necessary copies and distribute the revised plans to all affected parties.

1.10.6.2.2 Revisions Due to Minor Design Changes Requested by the Contractor

For revisions to the contract drawings required due to minor, acceptable modifications of the original design (e.g., change in prestressing of a prestressed concrete bridge beam) at the request of the contractor during construction, the contractor will be responsible for making the plan revisions. In such cases, the following procedure is to be used:

- (a) The District is to provide the plans of the contract drawing and copies of the relevant design computations to the contractor upon request.
- (b) The contractor must submit three sets of prints of the proposed revisions to the original drawings and three sets of supporting calculations of the modified design (if applicable), signed and sealed by a professional engineer licensed in the Commonwealth of Pennsylvania, to the District Bridge Engineer or his/her designated agent for review and acceptance. The design must be in complete compliance with the current editions of the AASHTO LRFD Bridge Design Specifications, PennDOT Publication 15M, Design Manual, Part 4, and the BD- and BC- standard drawings.

In addition, the following note and seal/signature block is to appear on the first sheet of revised contract drawings:

PLAN REVISION DESIGN	
Plan Revision R (insert revision number) encompasses a modification to the design of (insert modified component, e.g., “the prestressed concrete beams”). Sheets (list all affected sheets) and all related items.	
(Seal)	
(Signature)	Prepared by:

- (c) The shop drawing review procedure found in PP1.10.2.5 is to be followed with regards to acceptance of the plan revision by the Department.
- (d) Upon the Department’s acceptance of the proposed plan revisions, the contractor is to return the original plans, with the plan revisions made, to the District Office.
- (e) The District is to make the necessary copies and distribute the revised plans to all affected parties.
- (f) Any design computations required as part of the plan revision are to be amended to the original design computation package and recorded accordingly.

For a plan revision involving prestressed concrete bridge beams, the items to be addressed in the submission by the contractor include, but are not limited to, revisions of the strand pattern (e.g., fully-bonded straight, straight debonded or draped), debonding lengths, drape point locations, shear stirrup reinforcement bar size and spacing, beam cambers, beam daps, structure load ratings and beam seat elevations, as necessary and as applicable.

1.10.6.3 Acceptance of Plan Revisions

The District Bridge Engineer, his designated agent, or the original designer-of-record may accept the plan revision. Note, however, that all major structure revisions for Federal oversight projects must be processed through BDTD prior to revising the plans.

1.10.6.4 Drafting Procedures for Plan Revisions

The original plans are transmitted to Central Office as part of the PS&E package prior to bidding. Once a project has been awarded to the successful contractor, the plans are returned to the District Office for the purpose of reproducing and distributing the required number of copies to the appropriate parties.

Following receipt of the original plans from Central Office, make all revisions on the plans by crossing out erroneous or modified information, circling the correct information and marking the corrections with an R1, R2, etc. Follow standard drafting practices for linework, lettering, etc., as outlined in Design Manual, Part 3. Use the revision block, which is typically located above the title block to denote the nature of the revision, the initials of the responsible persons for drafting, checking and recommending (usually the District Bridge Engineer) the revision, as well as the recommendation date. Refer to Figure PP1.6.1-1 or PP1.6.2-1. If required to add additional sheets, locate within original set and designate as sheet #A, B, etc.

1.10.6.5 Microfilming and Storage Requirements

Refer to PP1.10.1.2.2a(d), (e), and (f) for microfilming and storage requirements for “as-built” plans.

1.11 STRUCTURE SUBMISSIONS – DESIGN-BUILD, LOW BID

1.11.1 General

In accordance with Publication 448 (Innovative Bidding Toolkit).

1.11.2 Conceptual Design

In accordance with Section 3.2.3 of Publication 448.

1.11.3 Final Design

In accordance with Section 3.3.4.4 of Publication 448.

1.11.4 Construction

1.11.4.1 Shop Drawings

In accordance with PP1.10.2.

1.11.4.2 Pile Hammer Approvals

In accordance with PP1.10.3.

1.11.4.3 Pile Load Test Evaluations

In accordance with PP1.10.4.

1.11.4.4 Construction Problems

In accordance with Section 3.3.4.2.7 of Publication 448.

1.11.4.5 Revisions During Construction and "As-Built" Plans

In accordance with applicable sections of PP1.10.6 and applicable sections of Publication 448, Chapter 3.

1.12 STRUCTURE SUBMISSIONS – DESIGN BUILD, ADJUSTED BID

1.12.1 General

In accordance with Publication 448, Chapter 4.

Editor's Note: The policy regarding the contracting type in Section PP1.12 is currently under review by the Department.

1.13 HAULING RESTRICTIONS AND PERMITS

1.13.1 General

Bridge members are designed in lengths, depths and widths that can be transported from the fabrication source to the project. Field splices, if required, are designed for specific locations shown on the structure drawings. The addition of field splices to shorten members, or the elimination of field splices to lengthen members, for shipping purposes, will not be approved on shop drawings without prior written approval by the District Bridge Engineer. In such cases, a sketch is required, and design computations prepared by a Registered Professional Engineer may be required. The Contractor assumes full responsibility for securing a hauling permit. Approval for elimination of a field splice at the shop drawing stage does not obligate the Department to issue a hauling permit. In either instance, no additional compensation to the Contractor will be allowed for the splice revisions.

1.13.2 Permit - Legal Loads

State laws controlling legal size loads restrict total length to 70 feet for "any LOAD non-divisible as to length hauled on a combination of vehicles". However, a non-divisible load can exceed 70 feet if a permit is obtained. For beams in excess of 110 feet in length, the following applies:

- (a) Special hauling permits may be issued for the movement on two-lane highways of overlength articles impossible to dismember, provided that the total length does not exceed 120 feet.
- (b) All movements having a total length in excess of 120 feet may be limited to multi-lane highways. Consideration shall be given only for short distances on two-lane highways (e.g., from the nearest port or railroad siding to the job site).
- (c) Movements longer than 160 feet total length or in excess of 201 000 pounds gross weight are defined as superloads in Department regulations (Title 67 Pennsylvania Code, Chapter 179). Approval for such movement may be secured only by submitting a preliminary application to the Central Permit Office for preliminary approval in accordance with Title 67 Pennsylvania Code, Chapter 179.8(4)(iii).

In the event that the design requires beams (non-divisible loads) longer than 110 feet, the designer shall make sure that the Central Permit Office will issue a permit for such a load. Therefore, for beams over 110 feet in length, the Designer shall include in the project special provisions any special hauling restrictions applicable to that particular design, thus alerting the Contractor to those requirements.

Travel over superelevated roads or bridges shall be weighed carefully, and needed lateral shoring to a deep beam shall be provided to avoid the possibility of overturning, which can jeopardize public safety.

1.14 SYSTEMS APPROVAL

Any proprietary system must undergo a Departmental evaluation and approval process prior to inclusion as an alternate system during the design phase or as a value engineering alternate during construction. The manufacturer must submit a New Product Evaluation Application (Form CS-4170) to the Innovation and Support Services Division. Instructions for completing the application for systems are included within the application. The New Product Evaluation Application is available at:

<ftp://ftp.dot.state.pa.us/public/PubsForms/Forms/CS-4170.pdf>

Do not permit any new wall system during construction.

The types of wall systems permitted for a project will be determined when the foundations are approved. When prefabricated walls are approved, the Contractor should be given an option to construct a conventional wall with the condition that secures a new foundation approval from the Department.

The Department bases its approval of the system and the supplier on the following considerations:

- (a) The system has a sound theoretical and practical basis for the engineer to evaluate and anticipate its claimed performance. For this purpose, the supplier or his representative must submit a package containing:
 - 1. The theory, its proponent, and the year it was proposed.

2. Where and how it was developed.
 3. Experiments supporting the theory.
 4. Field tests.
 5. Practical application, descriptions and photographs.
 6. A list of users including names, addresses, and telephone numbers.
 7. Names and qualifications of the designer(s). The Designer must be a Professional Engineer.
 8. Sample details of all elements, design calculations, strength and service limit state, factored force effects and nominal and factored resistance, estimated life, method of installation, and procedures for field and laboratory evaluation including instrumentation and special requirements, if any.
 9. Sample specifications showing material type, quality, certifications, fabrication requirements and method, field testing and acceptance criteria. Typical unit costs supported by data from actual construction.
- (b) If precast or prefabricated material is used in the system, the Precaster or Fabricator must be approved by the Department prior to approval for construction on Department projects.
- (c) The system will not be approved for frequent use unless it goes through the experimental installation and evaluation process.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 2 – SELECTION OF BRIDGE TYPES

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2.0 GENERAL

Selection of bridge type is a part of the type, size and location cost study which is based on foundation information obtained from a preliminary geotechnical engineering investigation, drainage conditions, highway limitations and environmental impact both present and future in accordance with PP1.9.3.

Unless other requirements (such as environmental commitments) govern, the most economical bridge type shall be selected. Lowering the design criteria is not acceptable. Minimum design live loads are specified in Design Manual, Part 2, Section 1.2. Therefore, low cost can only be achieved by careful selection of details and method of construction. Short spans and low cost bridges are reflected in various standards. The minimum number of spans, joints and beams should always be used. All efforts shall be made to minimize skew as much as and whenever possible.

Unless disapproved by the Deputy Secretary for Highway Administration, alternate designs by contractors are permitted in all cases. Whenever alternate designs are not allowed, justification must be provided.

Waterway permit requests shall include a permit for a temporary bridge or a temporary crossing for construction where applicable.

Structure-related environmental commitments shall be carefully considered and justified. Consideration shall be given to citizens, groups or other interest groups which, during the environmental clearance phase, may demand restricting a structure to only one material type. In these instances, the project manager must offer industry representatives the opportunity to develop and present alternate material concepts to the public for consideration. The Department's policy is to allow a contractor alternate design for bridges in every possible situation. Depicting only a steel or a concrete or a timber structure in public meetings or environmental documents could jeopardize the alternate design/bid policy. Design options chosen for display on graphics should be labeled, "preliminary, subject to change" or other similar wording. In a situation where a commitment may be appropriate, District Bridge Engineer's input shall be obtained prior to committing a bridge type. Keep all possible options open for the designer to provide a structurally sound and economical bridge design during the final design phase. Where it is necessary to deviate from the established standards, justifications and special approvals should be well documented.

Where practical, do not commit to stone lining. Preferable alternates are colored concrete and/or texture developed by form liners. Stone lining not only restricts structural inspection of the bridge members, but also is costly to construct and maintain.

When form liners are used, Table 2.0-1 provides guidance on selecting the most appropriate and economical form liner for various applications. Additionally, when form liners are required for abutments or retaining walls, the form liner shall extend to the full limits of the exposed concrete (where appropriate). Form liners shall not be terminated at corners, horizontal break points or proposed slopes with a smooth concrete boarder. The use of boarders around the perimeter of a form liner on an abutment or retaining wall shall not be specified due to additional cost and lack of historic context.

Table 2.0-1 – Form Liner Selection Guide

Features / Bridge Project Type	Form Liner Type Required ⁽⁴⁾		
	No form liner required	Small Reveal, $\leq 1''$ ⁽¹⁾	Deep Reveal, $> 1''$ ⁽²⁾
Scenic Byways	✓	✓	-
Scenic Rivers	✓	✓	-
Historic District	-	✓	✓
Historic Bridges	-	-	✓
Normal Project	✓	✓ ⁽³⁾	-

(1) Plastic construction and capable of being re-used between 20 and 30 times

(2) Elastomeric Urethane construction and capable of being re-used between 80 and 120 times

(3) Context sensitive solution for project has resulted in the use of an aesthetic treatment for bridge

(4) Liner does not apply to traffic barrier and other traffic safety features. Architectural treatments that comply with NCHRP Report 554 Aesthetic Concrete Barrier Design may be considered

In the broad definition of prestressing of concrete members, it can indicate both pretensioning and post-tensioning of concrete members. However, in this manual, prestressing will imply pretensioning, unless otherwise noted.

2.1 STEEL BRIDGES

2.1.1 Typical Steel Bridges Used in Pennsylvania

- (a) Composite steel multi-I-beam
- (b) Composite steel multi-plate girder

2.1.2 Restricted Types of Construction

Do not use pin-hanger and avoid piggy-back-type of construction.

2.1.3 General Requirements

Design bolted field splices in accordance with BD 616M. For lateral bracing requirements, see BD 620M.

2.2 CONCRETE BRIDGES

2.2.1 Reinforced Concrete Slab Bridges

This type of superstructure construction shall be used for widening of existing reinforced concrete slab bridges only when concrete box culvert or prestressed plank superstructure or concrete channel beam superstructure is not feasible or economical.

2.2.2 Reinforced Concrete T-Beam Bridges

This type of superstructure construction may be used for widening of T-beam bridges, but is not recommended for new construction. Even for widening, consider using precast channel or prestressed box beams since cast-in-place R.C. T-Beams widening would be costly unless the work is done by Department forces.

2.3 PRESTRESSED CONCRETE BRIDGES

2.3.1 Typical Prestressed Concrete Bridges in Pennsylvania

- (a) P/S adjacent box beam
- (b) P/S spread box beam
- (c) P/S I-beam
 - i. AASHTO Type
 - ii. PennDOT Type (preferred use for rehabilitations and widenings; not recommended for new structures)
- (d) P/S bulb-tee beam
- (e) NEXT beam
- (f) P/S segmental (with removable deck design and bonded post-tensioning features)

2.3.2 General Requirements

Unbonded post-tensioning system is not permitted for prestressed bridges, except as a corrective measure for existing bridges with the approval of the Chief Bridge Engineer.

Prestressed beam cross-sections must be selected from Standard Drawing BD-652M. Beam cross-sections and section properties that deviate from the current standards will require specific approval from the Chief Bridge Engineer.

Some longer span P/S I-beams and P/S bulb-tee beams will be in the superbeam category as defined in PP1.13.2. The use of superbeams should be investigated during the TS&L stage and approval for their use must be obtained from the Chief Bridge Engineer. Use of superbeams is permitted in alternate designs by the Contractor if it is not restricted by contract, all hauling restrictions are obeyed and a hauling permit can be issued by the Department.

With approval of the Chief Bridge Engineer, a P/S beam fabricator may be permitted to decrease the size of box beam void to simplify fabrication or to increase the section modulus of a beam used for structures with limited underclearance, or reduce lines of beams (beds) during fabrication. This may also be utilized in original designs where utilities or other requirements dictate maximum beam spacing, provided a thorough investigation of the special beam design has been made, the outside beam dimensions have not been altered, and the stresses are within allowable limits. In order to maintain design plan dimensions in box beams and to fabricate beams conforming to the tolerance requirements of P/S beams, fabricators are encouraged to specify 1/2 in.

undersize voids on the shop drawings. The additional 1/4 in. concrete around the inner perimeter of voids for box beams is not considered for section properties. However, the weight of this additional concrete shall be used in the original design.

2.4 TIMBER BRIDGES

The policy, procedure and criteria outlined in PennDOT Publication 9, "Policies and Procedures for the Administration of the County Liquid Fuel Tax Act of 1931 and Act 44 of 2007 and the Liquid Fuels Tax Act 655 dated 1956 and as amended", Publication 70, "Guidelines for the Design of Local Roads and Streets", and Publication 541, "Local Bridge Program Delivery Guide", all require that AASHTO and PennDOT bridge design criteria be used. Therefore, if Federal or State or liquid fuels funding is used in any part of the project, AASHTO and PennDOT bridge design criteria must be used. Timber bridge plans must be reviewed by BDTD before final approval.

Do not approve timber bridge plans without a review by the BDTD to ensure compliance with our design criteria and avoid omissions. A variety of timber bridges are being proposed and the design criteria of these bridges are in the developmental stage.

See A8 and D8 wood structures for design criteria.

Timber bridges may be used for:

- (a) Low-speed, low-truck-volume roads (ADTT less than 25 or ADT less than 750, whichever governs)
- (b) Locations where no debris exists for a condition where a timber pile pier is needed.
- (c) Locations where a timber pile substructure can be constructed economically, i.e., timber piles can be driven as friction piles to sufficient depth below potential scour depth and without being damaged, to get adequate lateral support. Generally, this is a problem because of the geology of Pennsylvania.

2.4.1 Typical Timber Bridges Used in Pennsylvania

- (a) Glulam - hardwood bridges
- (b) Other types if approved by the Chief Bridge Engineer.

2.4.2 Geometry

Bridge width shall be in accordance with PP3.1.1.

One-lane bridges may be permitted on local roads with PMC approval if no Federal funds are used, if advanced posting of one-lane bridge is provided, if the bridge width is not less than the approach pavement width, and if the owner absolves the Department from any legal responsibility resulting from the narrow width of the bridge.

2.5 CULVERTS

The following culvert types are generally used in Pennsylvania.

2.5.1 Metal Culverts

- (a) Corrugated steel/aluminum pipe culvert
- (b) Corrugated steel/aluminum pipe arch culvert
- (c) Steel/aluminum plate pipe culvert
- (d) Steel/aluminum plate pipe arch
- (e) Steel/aluminum box culverts

2.5.2 Precast Concrete Culverts

- (a) R.C.C. pipes
- (b) R.C.C. elliptical pipes
- (c) R.C.C. box culverts

2.5.3 Cast-in-Place Box Culverts

One-cell or multiple cells

2.5.4 Cast-in-Place Frame Structures

Use when unyielding foundation is available.

2.5.5 Precast Concrete System

- (a) CON/SPAN Bridge System
Permitted on NHS provided the precast units are post-tensioned
- (b) Bebo Precast Arch Bridge System
Permitted on non NHS.

2.6 OTHER BRIDGE TYPES

2.6.1 Unusual Bridges

- (a) Steel Box Girder
- (b) Through or deck truss
- (c) Steel or concrete arch
- (d) Cable-stayed
- (e) Tied-arch
- (f) Suspension
- (g) Two girder system (Permitted only in special condition or as a short-term temporary bridge)
- (h) Rigid Frame

2.7 SELECTION OF BRIDGE TYPES

A. Small bridges (up to 50 ft. span)

- 1. Single or multi-unit culvert
- 2. Slab
- 3. Composite concrete channel beam
- 4. R.C. T-beam (rarely used)
- 5. Timber
- 6. Composite P/S concrete plank beam
- 7. Composite P/S concrete adjacent box beam
- 8. Composite P/S concrete spread box beam
- 9. Composite P/S concrete I-beam or bulb-tee beam
- 10. Composite NEXT beam
- 11. Composite steel multi-I-beam
- 12. Composite steel multi-plate girder (generally for larger spans)

B. Medium size bridges (up to 165 ft. span)

- 1. Composite P/S concrete adjacent box beam
- 2. Composite P/S concrete spread box beam
- 3. Composite P/S concrete I-beam or bulb-tee beam
- 4. Composite steel multi-I-beam
- 5. Composite steel multi-plate girder
- 6. Deck and through steel truss (rarely used for this span range for new construction)
- 7. Rigid Frame

- C. Large Span Bridges (from 165 ft. to 500 ft. span)
 - 1. Composite steel multi-plate girder
 - 2. P/S concrete segmental (Removable deck and internally bonded design feature must be included)
 - 3. Composite Spliced P/S concrete girder
 - 4. Concrete arch
 - 5. Steel arch
 - 6. Steel deck truss
 - 7. Steel through truss
 - 8. Steel box girder (Generally not permitted)

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1

PART A: POLICIES AND PROCEDURES

CHAPTER 3 – DESIGN CONSIDERATIONS

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3.1 BRIDGE GEOMETRY

Considerations for bridge geometry shall take into account issues of highway safety, including sight distance, adequate clearances, and bridge widths compatible with the approach roadway.

3.1.1 Bridge Width

Unless otherwise directed, bridge width shall conform to Design Manual, Part 2, Chapter 1, Section 1.2. This includes criteria for bridges on Very Low Volume Roads.

3.1.2 Vertical Clearance

See D2.3.3.2.

3.1.3 Lateral Clearance

See D2.3.2.2.1.

3.2 SUPERSTRUCTURE

3.2.1 Girder Bridges

Girder bridges shall have a minimum of four girders unless approved otherwise by the Chief Bridge Engineer. The girder (beam) spacing shall be selected for optimum economy. Maximum girder spacing shall be 15 ft.. Four girders provide redundancy in the superstructure and will facilitate possible future redecking operations by allowing maintenance of traffic on one-half of the bridge.

A three-girder bridge with more than one traffic lane and without a floor system (floorbeams and stringers) is prohibited, except that a three-girder system for prestressed I-beam, bulb-tee beam or box beam superstructures may be permitted.

For three or more girders, the lateral live load distribution factors may be used as described in A4.6.2.2 and D4.6.2.2. In this case, the provisions of A3.6.1.1.2 and D3.6.1.1.2 (the multiple-lane presence factors) are already included in the factors and shall *not* be applied separately. When the lever rule is used for a three girder system, the multiple presence factor, as specified in A3.6.1.1.2, shall be applied.

In the special case where a two-girder system has been approved for design by the Chief Bridge Engineer, the fraction of live load distributed to each girder line shall be calculated by placing the loads on the bridge and summing moments about the opposite girder line. The provisions of A3.6.1.1.2 regarding multiple lanes of live load shall apply in this calculation.

3.2.2 Skew Angle

PennDOT defines skew angle as the smaller angle between the highway centerline (or a tangent thereto) and a line parallel to the support (wall, abutment, pier, etc.) or to the centerline of culverts (see Figure 3.2.2-1). AASHTO defines skew angle as the angle between the centerline of a support and a line normal to the roadway centerline. The sum of PennDOT's and AASHTO's skew angle is 90°, i.e.,

$$\theta_{PA} = 90 - \theta_{AASHTO} \quad (3.2.2-1)$$

$$\theta_{AASHTO} = 90 - \theta_{PA} \quad (3.2.2-2)$$

Except where noted, the AASHTO definition shall be used with AASHTO Specifications, and the PennDOT definition shall be used in all PennDOT documents and correspondence with the Department.

For steel bridges whose skew angle is less than 70°, the Engineer must submit, with the scope of work, the method to be used to analyze the structure. The analysis method selected will be part of the technical review and should be justified in the scope of work.

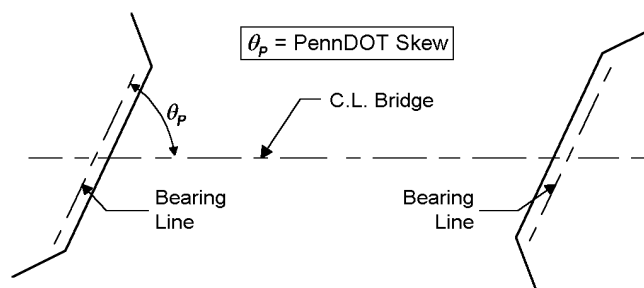


Figure 3.2.2-1 – PennDOT Skew Angle

3.2.3 Superstructure Drainage

3.2.3.1 Scupper Location

Scuppers shall be provided where hydraulic computations show that they are needed. Type 1 or Type 2 scuppers (see applicable BC Standards) may be provided at a minimum spacing of 50 ft. for the purpose of removing saltwater and anti-skid material on flat grades less than 2%. Scuppers shall also be provided at an approximate spacing of 400 ft. for long bridges with a profile grade of 2% and over, even if they are not needed hydraulically. Type 2 scuppers shall be used only if Type 1 scuppers cannot be accommodated.

For decks supported on adjacent prestressed concrete beams, the same criteria apply, except that standard rectangular metal curb drains shall be used. For decks with sidewalks supported on prestressed adjacent box beams, downspouting through box beams may be considered in such a way that the downspouting does not adversely affect the strand pattern. For decks supported on spread box beams, either Type 1 or Type 2 scuppers or curb drains may be provided, depending upon the overhang. Metal curb drains shall not be used on decks supported on steel or prestressed concrete I-beams or prestressed bulb-tee beams to avoid collection of contaminants on bottom flanges.

Free-falling scuppers shall be spaced so that the discharge is not adjacent to a substructure unit. Scuppers shall be omitted or the spacing shall be adjusted where the discharge would fall on roadways, railroad tracks, walkways, unprotected embankments, or other developed or highly erodible areas, or may be omitted entirely where their location outside the fascia beams is aesthetically unacceptable.

Flat pan scuppers should not be used because they collect debris and anti-skid material, and grow vegetation.

3.2.3.2 Scupper Types

When hydrologic and hydraulic calculations indicate the need for deck inlets, Type 1 scuppers shall be used exclusively. Since the hydraulic capacity of Type 2 scuppers is negligible, they should not be used where scuppers are needed hydraulically.

The Type 2 scuppers detailed in Standard Drawing BC-751M shall not be used unless approved by the Chief Bridge Engineer. If Type 2 scuppers are permitted, they shall be used with 8 in. diameter downspouting to eliminate debris accumulation and subsequent vegetative growth, and to ensure the downspouting capacity is greater than the scupper interception capacity.

3.2.3.3 End Structure Drainage

Roadway inlets shall be placed off the structure at the low end(s) of bridges. Inlets shall also be placed at the high end if the approach roadway is in cut, if the possibility of drawing significant roadway water over the bridge exists, or if the roadway is curbed. These facilities shall be included with the roadway pay items and shall be shown both on bridge design drawings and on roadway plans. On the bridge drawings, these inlets shall be indicated as Roadway Pay Items.

Locating a bridge in a sag curve or in a flat grade less than 0.5% shall be avoided to the maximum extent possible, since such geometry causes clogging of inlets. If such a location cannot be avoided, the roadway runoff shall be intercepted before it reaches the bridge.

3.2.3.4 Downspouting

Bridge scuppers shall be located to avoid long pipe runs. Pipe bends less than 135° shall be avoided when possible. For pipes sloped less than 45°, cleanout plugs shall be provided to ensure adequate maintenance. Downspouts shall be located preferably on the face of the pier which has the least exposure to the public view, but in no case shall downspouts interfere with specified vertical

and horizontal clearances. Downspouts shall not be embedded in pier stems, pier columns, abutments, wingwalls, or retaining walls. Redundant systems may be utilized at the discretion of the Bridge Engineer. Longitudinal runs shall not extend below the superstructure. A minimum slope of 8% shall be provided for longitudinal pipes between scuppers or from scuppers to point of discharge. Downspouting for free-fall condition shall be extended 6 in. below the adjoining beam. Refer to Standard Drawing BC-751M when designing downspouting for bridge drainage.

3.2.3.5 Splash Block

When discharge into storm drains is not practical or available, cast-in-place concrete blocks (splash blocks) [Refer to BC-751M] shall be placed in finished ground below the bottom of the vertical bridge drain pipe (downspout). The quantity of concrete shall be included in Class A cement concrete. Elevation at the top of the splash block at the centerline of the bridge drainpipe shall be shown on the drawings. The splash block shall be contoured and dished to contain and direct the flow away from substructure unit.

3.2.3.6 Drainage for Rehabilitation Projects

Treatment of bridge drainage in rehabilitation projects shall be as follows:

- (a) Existing flat pan scuppers shall be replaced with Type 1 scuppers in accordance with the criteria in PP3.2.3.1 and PP3.2.3.2. Flat pan scuppers shall be eliminated or their number reduced wherever possible. Existing flat pan scuppers may be retained only if Type 1 scuppers cannot be used.
- (b) If drains can be located near simple supports of a span, modification of the steel beam flanges may be considered to accommodate Type 1 scuppers. In some instances, turning a Type 1 scupper by 90° with the grate remaining parallel to the flow of water, or offsetting the pipe on one side of the scupper, may also be considered to avoid interference with the flanges.
- (c) For bridges rehabilitated using safety curbs and barriers:

If the existing curb-to-curb width is adequate, the same width should be maintained and overhang cut back to keep the same gutter lines for scuppers, if possible.

If existing scuppers must be retained and existing shoulders are widened by 6 in. using the safety curb and barrier, the deck shall be finished to drain into the scuppers. The grate seat shall be adjusted as needed.

3.2.3.7 Drainage System Design (Hydrologic and Hydraulic Calculations)

The following items are general comments on the articles in this section:

- The scupper interception capacity formula is specified on the basis of the updated information included in FHWA Hydraulic Engineering Circular No. 22.
- Scupper spacing procedures are specified to incorporate the procedures suggested in the FHWA Report Bridge Deck Drainage Guidelines and in FHWA Hydraulic Engineering Circular No. 22.
- PP3.2.3.7.2 permits water accumulation over the pavement of a maximum 2 ft. width. Generally, vehicles travel beyond or about that line. This practice is permitted for narrow shoulders to utilize practically unused pavement for water and/or snow storage.

The superstructure drainage shall be designed as follows:

3.2.3.7.1 Scupper Types

Details for various types of scuppers are shown on appropriate standard drawings.

3.2.3.7.2 Scupper Location

The spacing of scuppers shall be based on the following criterion:

The width of flow in the gutter for a ten-year frequency, five-minute duration, shall not exceed the width of shoulder. (For shoulders less than 6 ft. wide, a portion of the adjacent traffic lane, up to a maximum of one-fifth of the lane, may be included in the width of flow, if directed by the Department.)

3.2.3.7.3 Notation

- QR = Maximum runoff on bridge deck (cfs)
- Q_n = Maximum gutter storage capacity (cfs)
- Q_i = Scupper interception capacity (cfs)
- n = Roughness coefficient of deck surface = 0.016

3.2.3.7.4 Values of QR, Q_n and Q_i

(a) Value of QR

$$QR = \frac{CIA}{43,560} \quad (3.2.3.7.4-1)$$

where:

- C = Runoff factor = 0.90 for bridge decks
- I = Rate of rainfall for a given storm (in/hr). (This rate shall be determined in accordance with Publication 584, *PennDOT Drainage Manual*, Section 7.5.D)
- A = Drainage area (ft²) = WL, in which W and L are the width and length of deck to be drained by the scuppers, both in feet

(b) Value of Q_n

The maximum gutter storage capacity shall be determined assuming the full shoulder as the width of gutter (see Figure 3.2.3.7.4-1). Given the roadway grade (slope of channel) and the shoulder cross slope, determine $Q = Q_n$ from Figure 3.2.3.7.4-2.

where:

- S_x = Cross slope of deck or deck shoulder
- S = Grade as a function of location on bridge
- T = Design spread (ft.)

(c) Value of Q_i

The scupper interception capacity of the grate inlet on continuous grades shall be computed from the following relationship:

$$Q_i = (E)(Q) = Q[(R_f)(E_0) + R_s (1 - E_0)] \quad (3.2.3.7.4-2)$$

where:

- Q_i = Intercepted flow (cfs)
- E = Grate efficiency, dimensionless

Q = Total gutter flow (cfs)

R_f = Grate inlet frontal flow interception efficiency, dimensionless (see Figures 3.2.3.7.4-2, 3.2.3.7.4-3 and 3.2.3.7.4-4)

R_s = Grate inlet side flow interception efficiency, dimensionless (see Figure 3.2.3.7.4-5)

E_o = Ratio of front flow to total gutter flow, dimensionless (see Figure 3.2.3.7.4-6)

When using Figure 3.2.3.7.4-4, use curve P-1-7/8 for grate without cross bars. Where bicycle traffic is contemplated, cross-rods should be incorporated (P-1-7/8-4 curve) in the grate.

For further information on the above relationship, refer to Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual, Report No. FHWA-SA-96-078, November 1996.

The capacity of the 10 in. diameter downspout has been determined to be much greater than the scupper interception capacity and need not be calculated.

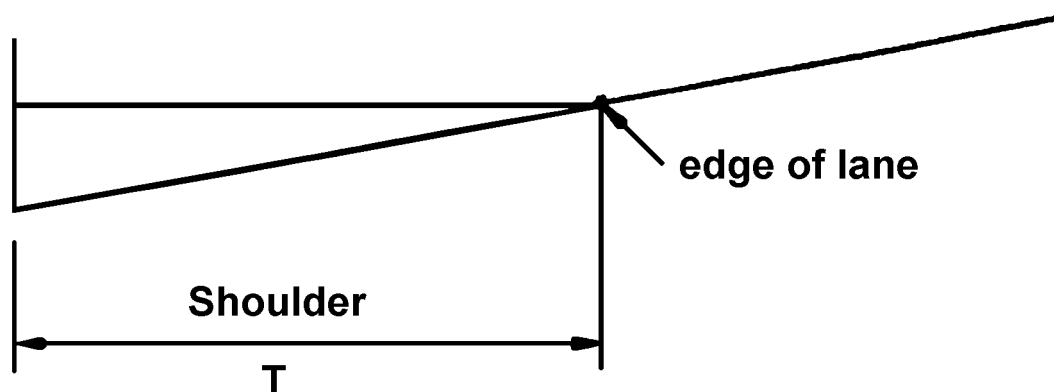


Figure 3.2.3.7.4-1 – Maximum Gutter Storage Capacity

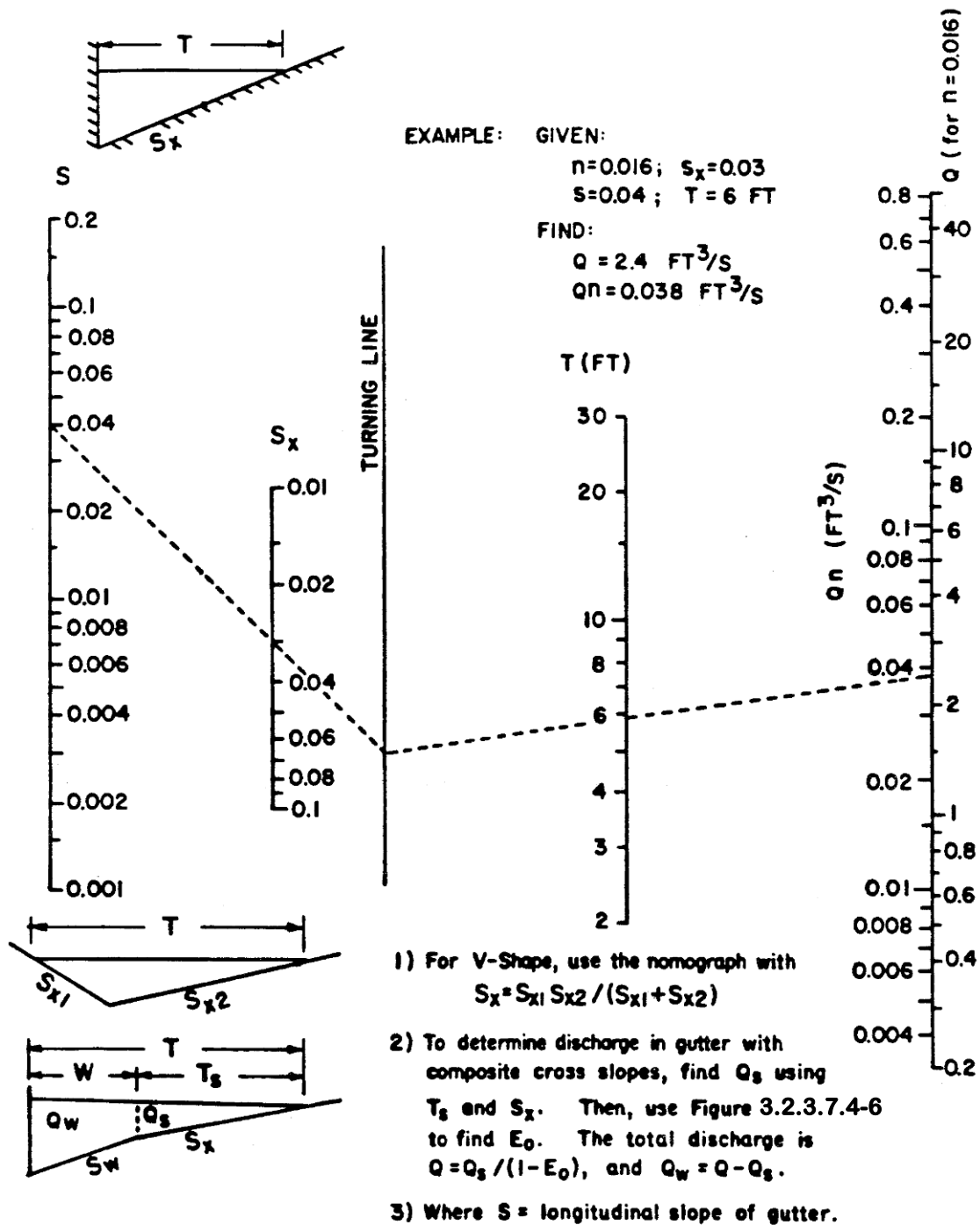


Figure 3.2.3.7.4-2 – Flow in Triangular Gutter Sections

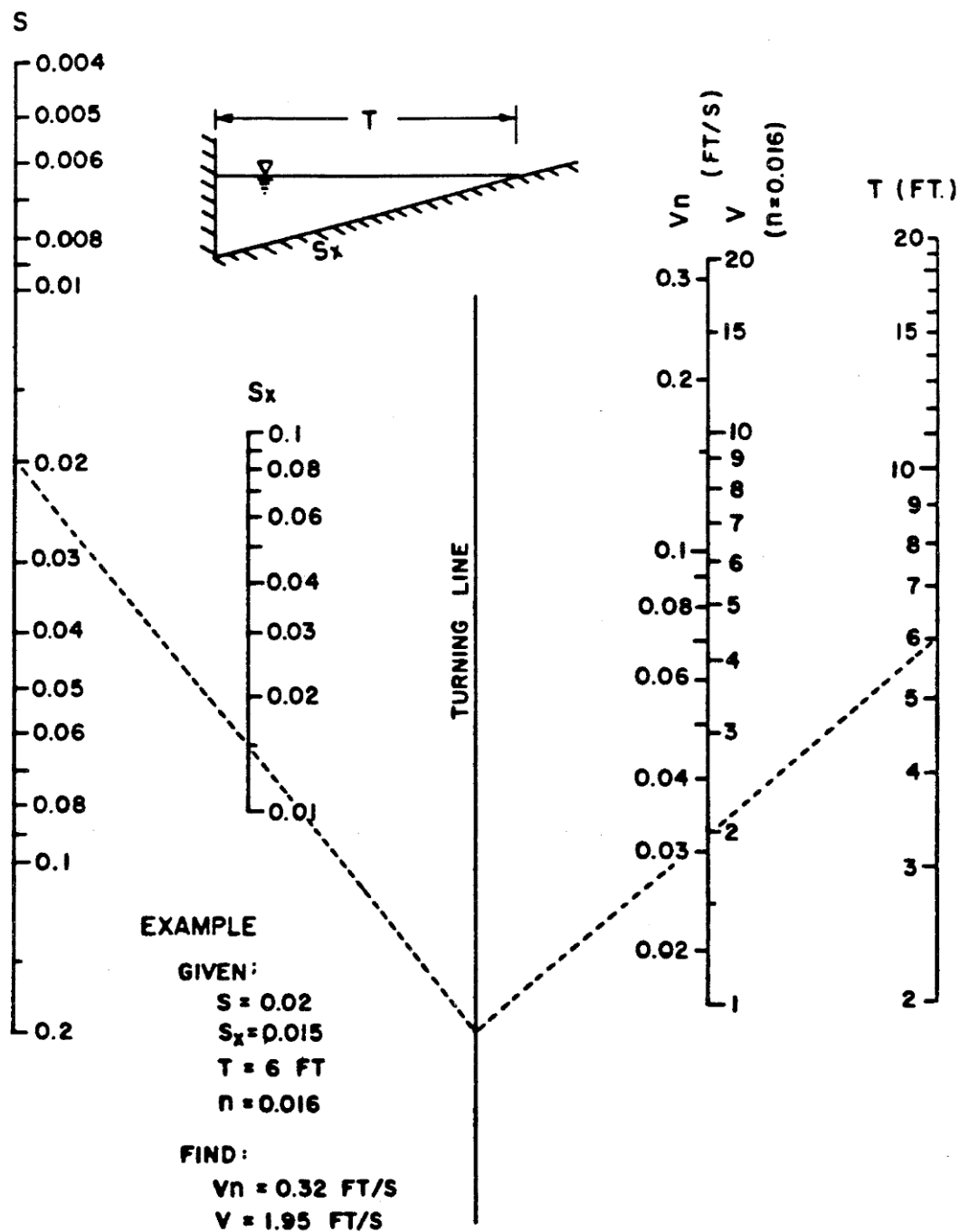


Figure 3.2.3.7.4-3 – Velocity in Triangular Gutter Sections

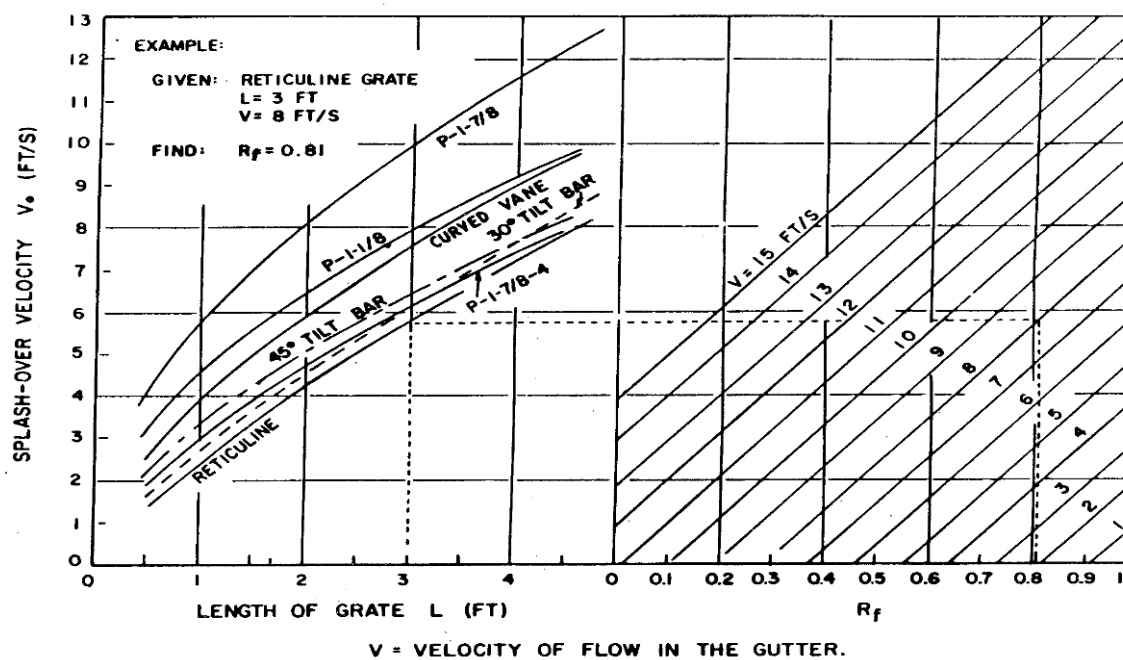


Figure 3.2.3.7.4-4 – Grate Inlet Frontal Flow Interception Efficiency

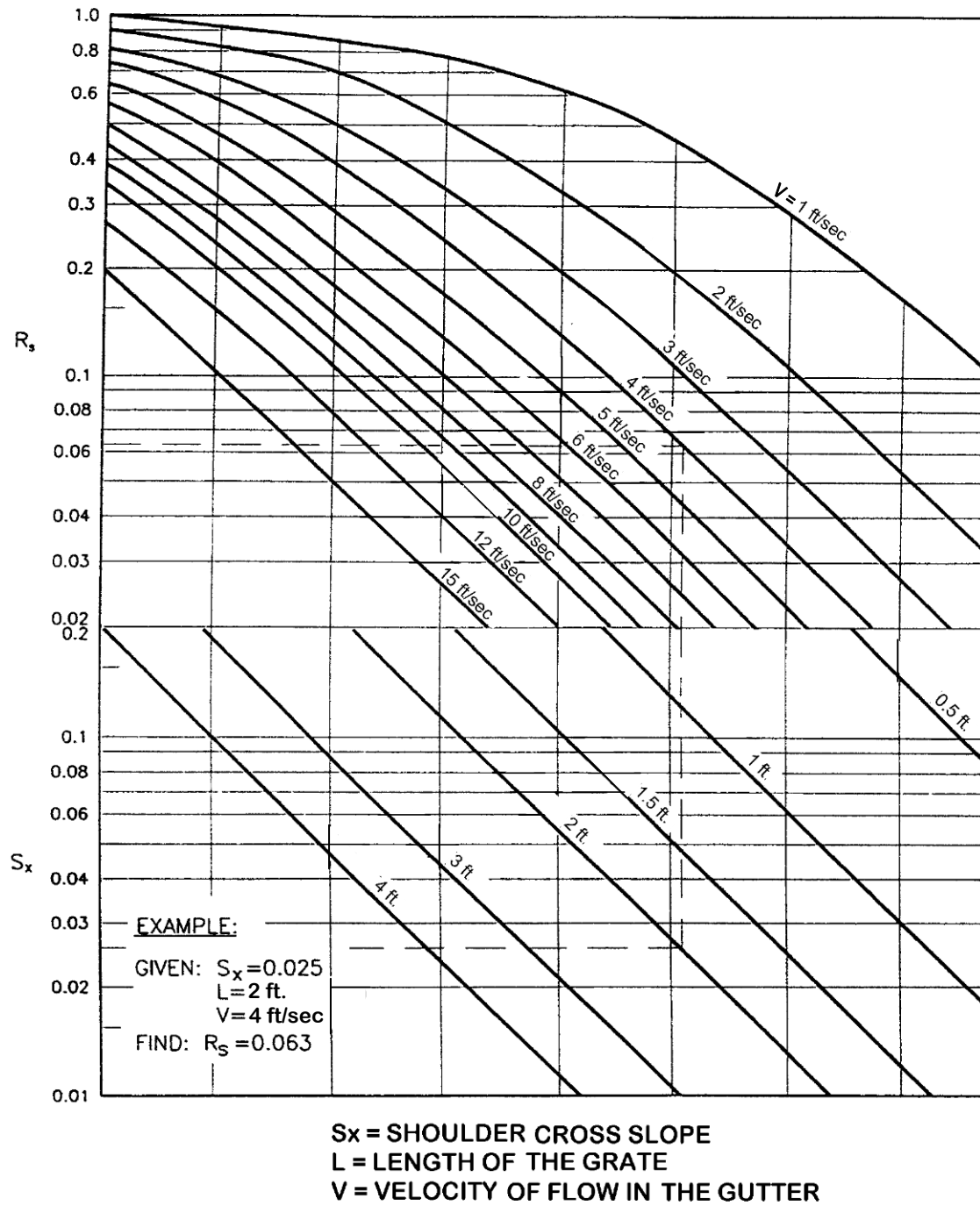
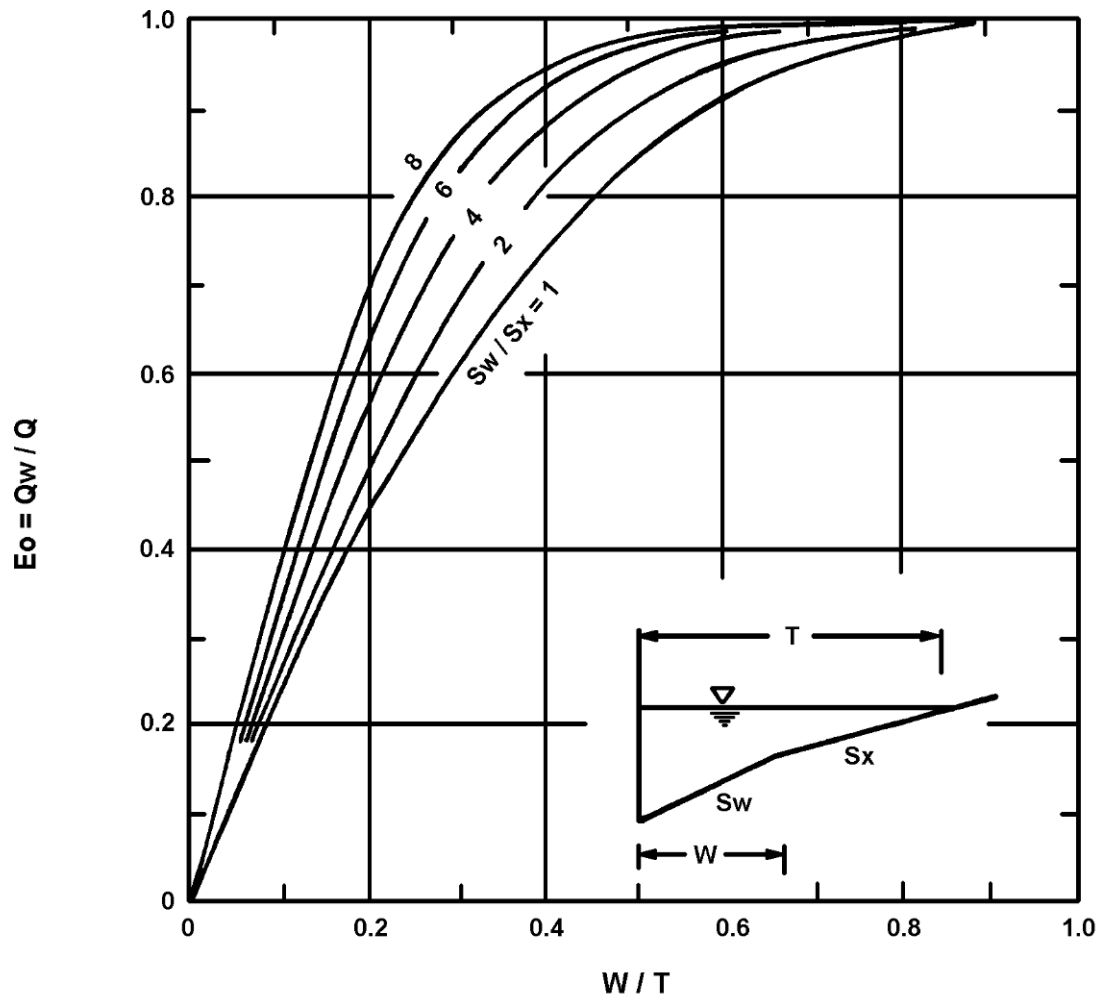


Figure 3.2.3.7.4-5 – Grate Inlet Side Flow Interception Efficiency



T = PERMITTED SPREAD WIDTH

W = GRATE WIDTH

Figure 3.2.3.7.4-6 – Ratio of Frontal Flow to Total Gutter Flow

3.2.3.7.5 Determination of Allowable Length of Bridge Without Scuppers

The allowable length of bridge without scuppers, L shall be determined by the use of Equation 3.2.3.7.5-1.

$$L = \frac{24,393.6}{C n} \left(\frac{S_x^{1.67} S^{0.5} T^{2.67}}{I W} \right) \quad (3.2.3.7.5-1)$$

where:

- C = 0.9 for bridge decks
- n = Roughness coefficient of deck surface = 0.016
- S_x = Cross slope of deck
- S = Grade as a function of location on bridge
- T = Design spread (ft.)
- I = Rate of rainfall (in/hr)
- W = Width of deck to be drained (ft.)

If the value of L as determined from Equation 3.2.3.7.5-1 is greater than the length of the bridge deck, scuppers are not needed hydraulically.

3.2.3.7.6 Scupper Spacing on Continuous Grades

If L, as determined from PP3.2.3.7.5, is less than the bridge deck length, scuppers are necessary. The following procedure for determining spacing is theoretical. Practical consideration may lead to constant spacing. The theoretical spacing may be revised for ease of placement, maintenance, etc.

- (a) Determine the distance from the high point to the first scupper (by trial and error if the bridge is within a vertical curve):

The distance, L, from the high point to the first scupper can be determined from Equation PP3.2.3.7.5-1.

- (b) Determine the distance from the first scupper to the second scupper:

- (1) Determine the quantity of the intercepted flow at the first scupper (q_1). (This location has grade S_1 .) Solve for the total gutter flow:

$$QR_1 = \frac{CIWL}{43,560} \quad (3.2.3.7.6-1)$$

where:

- I = Rate of Rainfall (in/hr)
- L = Distance from high point to first scupper (ft.)
- C = 0.9 for bridge decks
- W = width of drained deck (ft.)

Solve for the intercepted flow:

$$q_1 = (E) (QR_1) \quad (3.2.3.7.6-2)$$

in accordance with PP3.2.3.7.4, item (c).

- (2) Select M_1 , the distance from the first to the second scupper. (This establishes S_2 as the grade at $L + M_1$.)

- (3) Find total gutter flow at $L + M_1$:

$$QR_2 = \left(\frac{CIW}{43,560} (L + M_1) \right) - q_1 \quad (3.2.3.7.6-3)$$

- (4) Determine the gutter flow by using the nomogram in Figure PP3.2.3.7.4-2.

- (5) Find actual spread width, t , by using Figure PP3.2.3.7.4-2 to find this spread.

- (6) If t is equal to the permitted spread width T , the spacing is right. (If t is less than T , increase M_1 . If t is greater than T , decrease M_1 . Repeat the procedure above until the spacing is right.)

- (c) Determine the distance from the second scupper to the third scupper:

- (1) Determine the quantity of the intercepted flow at the second scupper q_2 . (This location has Grade S_2 .)

Use QR_2 computed from the preceding run. Solve for the intercepted flow:

$$q_2 = (E) (QR_2) \quad (3.2.3.7.6-4)$$

in accordance with PP3.2.3.7.4, item (c).

- (2) Select M_2 , the distance from the second to the third scupper. (This establishes S_3 as the grade at $L + M_1 + M_2$.)

$$(3) \quad QR_3 = \left(\frac{CIW}{43,560} (L + M_1 + M_2) \right) - q_1 - q_2 \quad (3.2.3.7.6-5)$$

- (4) Determine the gutter flow by using the nomogram in Figure PP3.2.3.7.4-2:

- (5) Find actual spread width, t , by using Figure PP3.2.3.7.4-2 to find this spread.

- (6) If t is equal to the permitted spread width T , the spacing is optimum. (If t is less than T , increase M_2 . If t is greater than T , decrease M_2 . Repeat the procedure until these values are equal.)

- (d) Determine the distance from the third to the fourth, the fourth to the fifth scupper, etc.

Continue with M_3 , M_4 , etc. until L , plus the sum of the spacing, equals the bridge length.

For detailed example problems and further information on the above spacing procedure, refer to the FHWA Report Bridge Deck Drainage Guidelines.

3.2.3.7.7 Spacing of Scuppers in a Sag

Scuppers shall be required in a sag vertical curve, one at the low point and one on each side of this point, where the grade elevation is approximately 0.20 ft. higher than that at the low point. The two adjacent scuppers (one at each side of the low point) are to be provided for contingency and not for hydraulic reasons. The capacity of the scupper at the low point shall be computed by neglecting the existence of the two adjacent scuppers.

The capacity of scuppers in a sag can be calculated from the following formula, assuming 50% efficiency:

$$Q_i = 0.5 (C_w) (P) (d)^{1.5} \quad (3.2.3.7.7-1)$$

where:

P = Perimeter of the grate (ft.), disregarding bars and the side against the curb

C_w = 3.0

d = Depth of flow at the curb (ft.)

A grate inlet in a sag location may operate as a weir or an orifice, depending on the grate properties (bar configuration and size) and depth of flow. Grates of larger dimension and grates with more open area, i.e., with less space occupied by lateral and longitudinal bars, will operate as weirs to greater depths than will smaller grates or grates with less open area.

Equation 3.2.3.7.7-1 applies to the weir condition, where shallow water depth generally governs. This equation should be adequate for the determination of the scupper capacity at a low point, since the allowable water depth for the superstructure drainage is generally shallow.

For further information regarding the interception capacity of inlets (scuppers) in sag locations, refer to FHWA Hydraulic Engineering Circular No. 22.

A suggested procedure for determining the actual spread width and the necessity of additional scuppers is indicated as follows:

(a) Determine Q_i (which equals QR) from the selected length and width of the bridge deck to be drained, in accordance with PP3.2.3.7.4, item (a).

(b) Determined from:

$$d^{1.5} = \frac{Q_i}{0.5(C_w) P} \quad (3.2.3.7.7-2)$$

(c) Obtain the actual spread width, t, using d computed as above and the known gutter cross slope.

(d) If t is equal to or less than the permitted spread width T, no additional scuppers are needed for the selected length from the hydraulic point of view. If t is greater than T, additional scuppers are needed.

3.2.4 Paving Notch and Bridge Approach Slab

Provide a paving notch or an attached approach slab on structures out-to-out on all interstates and expressways and all locations with concrete approach pavements. A paving notch is not required on structures with ADT less than 750 or when a concrete pavement is not anticipated in the future unless differential settlement between the approach fill and the structure are anticipated or providing an approach slab will eliminate a joint on the structure.

A bridge approach slab is not required on structures with ADT less than 750 unless providing an approach slab will eliminate a joint on the structure. However, approach slabs are always required with integral abutments and shall be the same curb-to-curb width of the bridge. The U-wings must be designed in such a manner as to permit approach slab movement independent of the U-wings.

3.3 SUBSTRUCTURE

3.3.1 Slope Walls

Slope walls under the end spans are required, when directed by the District Executive, to protect the slopes from erosion and to eliminate unsightly appearance of barren slopes where aesthetics is a factor or where maintenance is impractical. However, slope walls may not be necessary in rock cut areas.

Cement concrete slope walls, either precast or cast-in-place (see Standard Drawing BC-731M), and random stone slope walls (see Standard Drawing BC-781M) can be used alternatively. However, random stone slope wall is low in cost and suitable for use in rural areas where vandalism is not a great concern.

Stone slope wall may be used in scenic areas for aesthetic purposes (see Standard Drawing RC-40M).

3.3.2 Substructure Drainage

Underdrain pipes which extend from the end or from the face of walls to the highway drainage system shall be identified on the structure drawings as a Roadway Pay Item and shall be included in the Required List on the Roadway Plans.

3.3.3 Abutments and Cast-in-Place Retaining Walls

Abutments, wingwalls and retaining walls shall be of the reinforced concrete cantilever type, except for 100% state-funded projects, where, if specified, wingwalls and retaining walls of overall height of 12 ft. or less may be of the type shown on Standard Drawing BD-631M.

Use of approved proprietary walls, tie-back walls, or concrete cantilever walls shall be evaluated in light of economics and site suitability.

Approved alternate walls shall be permitted unless only one type is practical at a particular site. The wall types (on the approved list) that are not permitted for a site shall be indicated or specified in the contract.

3.3.4 Prefabricated Retaining Walls

3.3.4.1 General

Prefabricated walls, including approved proprietary systems, shall be used in competition with conventional reinforced concrete walls where conventional wall design is provided. If conventional walls are clearly not competitive, they shall be excluded from the alternates, thereby saving time and costs provided that two or more prefabricated systems are available.

Preferably none of the suppliers should be contacted during the design stage. However, if during the design of a project, it becomes necessary for the designer to obtain detailed information on any of the proprietary walls, the suppliers of all types permitted in the foundation approval shall be contacted and offered the same degree of involvement so as not to give a particular supplier a time advantage in preparing his or her bid.

3.3.4.2 Systems Approval

For systems approval guidelines and consideration, see PP1.14.

3.3.4.3 Selection Procedure

All feasible, innovative, cost-saving alternates must be considered as follows:

- (a) For use as an alternate system during design phase: Consider and permit all feasible alternates. If only one proprietary system is feasible, it shall be specified only as an alternate to a conventional design. Use of only one system must be justified and approval must be secured from the Chief Bridge Engineer before the system is specified.
- (b) Value engineering alternate during construction: Contractors may propose any cost-saving, equivalent, approved alternate, with adequate justification.
- (c) Experimental use: An experimental system will not be permitted as an alternate.

3.3.4.4 Economic Considerations for Project Selection

The decision to designate a prefabricated retaining wall for a particular project requires the determination of its technical feasibility and its economy compared with conventional construction. With respect to economy, the following guidelines are provided:

- (a) Mechanically stabilized earth (MSE) walls are generally more economical than conventional cast-in-place retaining walls in fill situations under the following conditions:
 - (1) Where the retaining wall has a total area greater than 2,000 ft²
 - (2) For average wall heights greater than 10 ft. with no traffic barrier
 - (3) For average wall heights greater than 15 ft. when traffic barriers are required

- (4) For walls of any height, where a rigid conventional wall system requires a deep foundation for support, provided that settlements of MSE walls are tolerable.
- (b) Concrete modular systems and soldier pile walls are generally more economical than conventional cast-in-place retaining walls in cut situations, where substantial excavation is necessary for average wall heights greater than 8 ft. and where the retaining wall area is greater than 500 ft²
- (c) Specific project conditions, as outlined below, may reduce the cost-effectiveness of prefabricated wall systems:
 - (1) Limited availability and high cost of select backfill
 - (2) Complicated horizontal alignment requiring many turning points and highly irregular finished grades
 - (3) The necessity of providing temporary excavation support systems during construction

3.3.4.5 Plan Preparation

Where prefabricated retaining walls are permitted as alternates, the conceptual design used for bidding purposes shall contain the following minimum information:

- (a) Beginning and end of wall stations
- (b) Elevations on top of wall at beginning and end of wall, all profile break points, and roadway profile data at wall line
- (c) Original and proposed ground profiles in front of and behind retaining wall
- (d) Cross-sections at retaining wall location, showing limits of excavation and backfill
- (e) Horizontal alignment of wall
- (f) Details of wall appurtenances such as traffic barriers, copings, drainage outlets, location and configurations of overhead signs, lighting including conduit locations, and all affected utilities
- (g) Right-of-way limits and all affected utilities
- (h) Construction sequence requirements, if applicable, including traffic control, access, and staged construction sequences (particularly for abutments); how much settlement must have occurred per stage, and any jacking requirements due to settlement
- (i) Elevation of bottom of wall and maximum allowable bearing capacity at this level for each type of wall; location, depth, and extent of any unsuitable material to be removed and replaced
- (j) Magnitude, location, and direction of external loads due to bridges, overhead signs, and lighting structures
- (k) Architectural treatment for facing panel or module finishes and colors
- (l) Quantities table showing estimated wall area, abutment and wingwall quantities, as well as appurtenances and traffic barriers
- (m) At abutments, elevation of bearing pads, location of bridge seats, skew angle, and all horizontal and vertical survey control data including clearances; complete design and details of stub abutment for MSE system. Show dead load, live load and longitudinal loads perpendicular to the abutment per linear foot on the plans for abutments on prefabricated walls
- (n) Limits and requirements for drainage features beneath, on top of, and behind retaining structure (Note: Drainage systems within the retained backfill material, shall be 100 year pipe per Design Manual, Part 2, including watertight joints)
- (o) At stream locations, extreme high-water and normal-water levels and scour protection

- (p) Governing construction specifications and special provisions
- (q) Limit for epoxy-coated bars in the panels (refer to D5.4.3.6P)
- (r) Estimated settlement, factored bearing resistance, and slope stability analysis where applicable (to be submitted with the foundation approval submission)
- (s) Reference to D11.10 governing design requirements and allowable deviations, if any
- (t) Foundation information (to be submitted with foundation approval submission): for a depth equal to 150 percent of the anticipated length of soil reinforcement at any wall location, for all soil strata
 - (1) Drained angle of internal friction, ϕ
 - (2) Undrained shear strength, C_u , for saturated cohesive materials
 - (3) Total density of the material
 - (4) Boring logs
 - (5) Location of watertable
 - (6) Random backfill information
 - Drained angle of internal friction
 - Cohesive strength
 - Total density

If the above information is not provided, the appropriate values given in D10 and D11 shall be used.

As a part of the foundation submission, the designer shall estimate the applied or design bearing pressure and compare it with the calculated factored bearing resistance. Where an abutment is supported by the prefabricated wall, pressure applied by abutments shall be included in the calculation of the total applied pressure. If the estimated applied pressure exceeds the factored bearing resistance, the designer shall consider such remedial measures as overexcavation and replacement with granular backfill and shall indicate in the plans the depth and lateral extent of the foundation modifications.

- (u) External stability, including overturning, sliding, settlement, and overall slope stability shall be checked using an approved analysis method. For analysis purposes, a base width equal to 0.7 of the height or the minimum specified width shall be used for mechanically stabilized earthwalls.
- (v) All MSE wall plans (if applicable) must show pile locations and proposed location and arrangement of MSE wall soil-reinforcing elements to avoid interference with the piles. In some instances, pile locations interfere with soil-reinforcing grids or soil-reinforcing strips behind the MSE walls. Specific method should be developed for the field installation to avoid and circumvent interference with piles. Simply cutting soil-reinforcing elements (grids or strips) is not acceptable.
- (w) For abutments, wingwalls and retaining walls, eliminate flexible open cell polyether or polyurethane foam strip fillers for horizontal joints between panels; cover all joints between panels on back side of the wall with geotextile Class 4, Type A fabric. Apply adhesive coating on panels only and not on geotextile fabric. Do not apply adhesive within 2 in. of the joint.

For Precast Modular Unit Retaining Walls, T-walls, and any prefabricated wall, provide complete drainage behind the wall and provide weepholes in the front of the wall as given in Standard Drawing BC-799M.

These procedures will provide free drainage and minimize hydrostatic pressure.

3.3.4.6 Proposal Preparation and Bidding Instructions**(a) Proposal Preparation**

Where prefabricated walls are used for retaining walls and conventional retaining walls are permitted as alternates, the Lump Sum bid items will include "Retaining Wall, S-xxxxx", "Mechanically Stabilized Retaining Wall,

S-xxxxxP" and "Precast Modular Retaining Wall, S-xxxxxP". Note that the same S-number will appear in all items.

When conventional retaining walls are NOT permitted as alternate designs, conceptual drawings for the prefabricated walls should be shown on the structure drawings with specific S-number. However, the Lump Sum bid items will include "Mechanically Stabilized Retaining Wall, S-xxxxxP" and "Precast Modular Retaining Wall,

S-xxxxxP".

Where prefabricated walls are not permitted as alternates for retaining walls, the lump sum bid items will include "Retaining Walls, As-Designed, S-xxxxx".

Prefabricated walls shown in accordance with PP3.3.4.5 are not considered as "as-designed" walls. Therefore, a detailed break down of the prefabricated wall quantities for the "Component Item Schedule" is not required. However, the successful bidder will be required to submit a "Component Item Schedule" for the prefabricated wall in accordance with the Special Provisions (Part A).

Figure 3.3.4.6-1 through Figure 3.3.4.6-6 are provided for your guidance. They cover the procedures for preparing the PS&E package, where the proposal includes prefabricated walls and alternate bridge structures. For typical tabulations of quantities, refer to Figures 3.3.4.6-7 through 3.3.4.6-13. Note that Figure 3.3.4.6-6 should be avoided, if conventional walls are clearly not competitive, by using Figure 3.3.4.6-5.

(b) Bidding Instructions

In order to give suppliers of prefabricated walls sufficient time to prepare bids, the presence of these items in forthcoming projects must be described in the D-407 (given in Design Manual, Part 2) cover sheet submittal, so that the necessary information can be included in the Notice to Contractors. This is especially important for prefabricated walls because designs of sufficient depth to permit reliable pricing must be developed by the suppliers during the advertising period.

When prefabricated walls are used for the wingwalls and/or abutments of a bridge, they will be considered as separate lump sum items, for both as-designed bridges and contractor-designed alternates. Only general layout and elevation information for the various prefabricated walls permitted will be shown for the as-designed bridge. The design criteria will be in accordance with this Manual or as modified in the Special Provisions.

The bidders will be required to indicate the type of prefabricated wall intended for construction. The Contractor shall submit, for approval, a design of the prefabricated wall as would be done for any contractor design alternate.

WHERE PREFABRICATED WALLS ARE USED AS RETAINING WALLS AND CONVENTIONAL R.C. WALLS ARE PERMITTED:

- ** ITEM 8610-00xx - CONCRETE RETAINING WALL, S-xxxxx
- * ITEM 8621-00xx - MECHANICALLY STABILIZED RETAINING WALL, S-xxxxxP
- * ITEM 8622-00xx - PRECAST MODULAR RETAINING WALL, S-xxxxxP

Construct one of the above at S.R. _____,
Section _____, Segment _____, Offset _____.

ALTERNATE WALL PART A: I-c80101-

Include the appropriate retaining wall system Standard Special Provisions, e.g. N-a11701-, N-a12501-.

- * List only applicable retaining wall systems.
- ** Include "Component Item Schedule" for as-designed wall

Figure 3.3.4.6-1 – Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Permitted

WHERE PREFABRICATED WALLS ARE USED AS RETAINING WALLS AND CONVENTIONAL R.C. WALLS ARE NOT PERMITTED (STRUCTURE DRAWINGS REQUIRED):

- * ITEM 8621-00xx - MECHANICALLY STABILIZED RETAINING WALL, S-xxxxxP
- * ITEM 8622-00xx - PRECAST MODULAR RETAINING WALL, S-xxxxxP

Construct one of the above at S.R. _____,
Section _____, Segment _____, Offset _____.

ALTERNATE WALL PART A: I-c80121-

Include the appropriate retaining wall system Standard Special Provisions, e.g. N-a11701-, N-a12501-.

- * List only applicable retaining wall systems.

NOTE: Refer to ECMS for the current document number of all Standard Special Provisions.

Figure 3.3.4.6-2 – Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Not Permitted

WHERE CONVENTIONAL R.C. WALLS ARE USED AS RETAINING WALLS AND PREFABRICATED RETAINING WALLS ARE NOT PERMITTED:

** ITEM 8610-00xx - CONCRETE RETAINING WALL, AS-DESIGNED, S-xxxxx

Construct the above at S.R. _____,
Section _____, Segment _____, Offset _____.

** Include "Component Item Schedule" for as-designed wall

Figure 3.3.4.6-3 – Conventional R.C. Walls are used as Retaining Walls and Prefabricated Retaining Walls are not Permitted

LUMP SUM BRIDGE STRUCTURE WITH CONVENTIONAL ABUTMENTS AND CONVENTIONAL WINGWALLS WHERE PREFABRICATED ABUTMENTS AND WINGWALLS ARE NOT PERMITTED:

** ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS-DESIGNED, S-xxxxx
ITEM 8100-00xx - STEEL BRIDGE STRUCTURE
ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE

Construct one of the above at S.R. _____,
Section _____, Segment _____, Offset _____.

ALTERNATE BRIDGE STRUCTURE: I-c80041-

Special Drawings and Special Design Requirements (PART B)

Piles (when as-designed bridge includes piles)

NOTE: Refer to ECMS for the current document number of all Standard Special Provisions.

** Include "Component Item Schedule" for as-designed structure.

Figure 3.3.4.6-4 – Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Walls are Not Permitted

LUMP SUM BRIDGE STRUCTURE WITH PREFABRICATED WALL ABUTMENTS AND/OR PREFABRICATED WINGWALLS WHERE CONVENTIONAL ABUTMENTS AND WINGWALLS ARE NOT PERMITTED (BEING UNECONOMICAL):

** ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS-DESIGNED, S-xxxxx
 ITEM 8100-00xx - STEEL BRIDGE STRUCTURE
 ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE

Construct one of the above at S.R. _____,
 Section _____, Segment _____, Offset _____.

ALTERNATE BRIDGE STRUCTURE: I-c80041-

Special Drawings and Special Design Requirements (PART B)

Piles (when as designed bridge includes piles)

NOTE:

- Provide a separate pay item in the schedule of prices for prefabricated abutments and/or wingwalls (do not provide separate S-number for prefabricated wall layout drawing).
- Stub abutments, structure backfill behind stub abutments should be included with bridge structure lump sum.

* ITEM 8641-00xx - MECHANICALLY STABILIZED ABUTMENT OR WINGWALL, S-xxxxxP

* ITEM 8642-00xx - PRECAST MODULAR ABUTMENT OR WINGWALL, S-xxxxxP

Construct one of the above at S.R. _____,
 Section _____, Segment _____, Offset _____.

ALTERNATE WALL PART A: I-c80121-

Include appropriate prefabricated wall system Standard Special Provisions, e.g., N-a11701-, N-a12501-.

NOTE: Refer to ECMS for the current document number for all Standard Special Provisions.

** Include "Component Item Schedule" for as-designed bridge structure.

* List only applicable retaining wall systems

Figure 3.3.4.6-5 – Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Prefabricated Wingwalls where Conventional Abutments and Wingwalls are Not Permitted

LUMP SUM BRIDGE STRUCTURE WITH CONVENTIONAL ABUTMENTS AND CONVENTIONAL RETAINING WALLS AND WINGWALLS WHERE PREFABRICATED ABUTMENTS AND/OR WINGWALLS ARE PERMITTED (WHEN CONVENTIONAL ABUTMENTS ARE COMPETITIVE):

NOTE: If possible, avoid this figure by using Figure 3.3.4.6-4. However, if included on PS&E package, use the following procedure:

- + ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS-DESIGNED (INCLUDES CONVENTIONAL ABUTMENT AND WINGWALL DESIGN), S-xxxxx
- ++ ITEM 8xxx-00xx - BRIDGE STRUCTURE, AS DESIGNED (DOES NOT INCLUDE CONVENTIONAL ABUTMENT AND WINGWALL DESIGN), S-xxxxx
- ITEM 8100-00xx - STEEL BRIDGE STRUCTURE (CONVENTIONAL ABUTMENTS)
- ITEM 8100-00xx - STEEL BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)
- ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE (CONVENTIONAL ABUTMENTS)
- ITEM 8000-00xx - PRESTRESSED CONCRETE BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)

Construct one of the above at S.R. _____,
Section _____, Segment _____, Offset _____.

ALTERNATE BRIDGE STRUCTURE: I-c80041-

Special Drawings and Special Design Requirements
(PART B).

Piles (when as-designed bridge includes piles)

ITEM 8640-00xx PREFABRICATED ABUTMENT OR WINGWALL, S-xxxxxP

Construct one of the following at S.R. _____,
Section _____, Segment _____, Offset _____:

- * 1. Mechanically Stabilized Retaining Wall System.
- * 2. Precast Modular Retaining Wall System.

ALTERNATE WALL PART A: I-c80121-

Include appropriate prefabricated wall system Standard Special Provisions, e.g. N-a11701-, N-a12501-.

NOTE: Refer to ECMS for the current document number of all Standard Special Provisions.

- + Include "Component Item Schedule" for as-designed structure
- ++ Include "Component Item Schedule" but reduce conventional abutment related quantities (Class 3 Excavation; Class A Concrete; Class AA Concrete; Reinforcement Bars; Structure Backfill Material and other items related to conventional abutments)
- * List only applicable retaining wall systems

Figure 3.3.4.6-6 – Lump Sum Bridge Structure with Conventional Abutments and Conventional Retaining Walls and Wingwalls where Prefabricated Abutments and/or Wingwalls are Permitted

When Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Permitted (Refer to Figure 3.3.4.6-1):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NUMBER	INFORMAL QUANTITY & UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
	UNIT							
EITHER LS	8610 0001		CONCRETE RETAINING WALL, S-XXXXX	---				
	LS							
		613 CY	CLASS 3 EXCAVATION	289				
		1, 217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640				
		393 CY	CLASS A CEMENT CONCRETE	179				
AND 31,563	1002 0053		REINFORCEMENT BARS, EPOXY-COATED	2,270				
	LB							
OR LS	8621 0002		MECHANICALLY STABILIZED RETAINING WALL, S-XXXXXP	(---) SF				
	LS							
OR LS	8622 0002		PRECAST MODULAR RETAINING WALL, S-XXXXXP	(---) SF				
	LS							

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-7 – Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Permitted

When Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are not Permitted (Refer to Figure 3.3.4.6-2):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS							
QUANTITY	ITEM NUMBER	DESCRIPTION	RETAINING WALL OR WINGWALL				
	UNIT						
EITHER LS	8621 0001	MECHANICALLY STABILIZED RETAINING WALL, S-XXXXXP	(---) SF				
	LS						
OR LS	8622 0002	PRECAST MODULAR RETAINING WALL, S-XXXXXP	(---) SF				
	LS						

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-8 – Prefabricated Walls are used as Retaining Walls and Conventional R.C. Walls are Not Permitted

When Conventional R.C. Walls are used as Retaining Walls and Prefabricated Retaining Walls are not Permitted (Refer to Figure 3.3.4.6-3):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NUMBER	INFORMAL QUANTITY & UNIT	DESCRIPTION	RETAINING WALL OR WINGWALL				
	UNIT							
EITHER LS	8610 0001	CONCRETE RETAINING WALL, AS DESIGNED, S-XXXXX		---				
	LS							
		613 CY	CLASS 3 EXCAVATION	289				
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640				
		393 CY	CLASS A CEMENT CONCRETE	179				
AND 31,563	1002 0053	REINFORCEMENT BARS, EPOXY-COATED		2,270				
	LB							

Figure 3.3.4.6-9 – Conventional R.C. Walls are used as Retaining Walls and Prefabricated Retaining Walls are Not Permitted

Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are Not Permitted (Refer to Figure 3.3.4.6-4):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO.	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
	UNIT							
EITHER LS	8010 0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX						
	LS							
		613 CY	CLASS 3 EXCAVATION	289	100	224		
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640	100	477		
		393 CY	CLASS A CEMENT CONCRETE	179	100	114		
		LS	STEEL BEAM TEST PILES, HP 12 X 53	2 @ 37'				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24' 1 @ 35'		
AND 31,563	1002 0053	REINFORCEMENT BARS, EPOXY-COATED		2,270		186	1,000	28,107
	LB							
AND 1,672	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53		910		762		
	LF							
AND 75	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		42		33		
	EACH							
OR LS	8100 0002	STEEL BRIDGE STRUCTURE						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53		(---)		(---)		
	LF							

Figure 3.3.4.6-10 – Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are Not Permitted

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO.	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
	UNIT							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT HP 12 X 53		(---)		(---)		
	EACH							
OR LS	8000 0003	PRESTRESSED CONCRETE BRIDGE STRUCTURE						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53		(---)		(---)		
	LF							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		(---)		(---)		
	EACH							

Figure 3.3.4.6-10 – Lump Sum Bridge Structure with Conventional Abutments and Conventional Wingwalls where Prefabricated Abutments and Wingwalls are Not Permitted (continued)

Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted (Refer to Figure 3.3.4.6-5):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO.	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
	UNIT							
EITHER LS	8010 0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX	---					
	LS							
		613 CY	CLASS 3 EXCAVATION	289	100	224		
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640	100	477		
		393 CY	CLASS A CEMENT CONCRETE	179	100	114		
		LS	STEEL BEAM TEST PILES, HP 12 X 53	2 @ 37'				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24' 1 @ 35'		
AND 31,563	1002 0053	REINFORCEMENT BARS, EPOXY-COATED	2,270			186	1,000	28,107
	LB							
AND 1,672	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53	910			762		
	LF							
AND 75	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	42			33		
	EACH							
OR LS	8100 0002	STEEL BRIDGE STRUCTURE						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53	(---)			(---)		
	LF							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)			(---)		
	EACH							

Figure 3.3.4.6-11 – Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO.	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
	UNIT							
OR LS	8000 0003	PRESTRESSED CONCRETE BRIDGE STRUCTURE						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53		(---)		(---)		
	LF							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		(---)		(---)		
	EACH							
LS	8641 0001	MECHANICALLY STABILIZED ABUTMENT OR WINGWALL, S-XXXXXP		(---) SF		(---) SF		
	LS							
OR LS	8642 0001	PRECAST MODULAR ABUTMENT OR WINGWALL, S-XXXXXP		(---) SF		(---) SF		
	LS							

(List Applicable Prefabricated Wall Systems on the Drawing)

Figure 3.3.4.6-11 – Lump Sum Bridge Structure with Prefabricated Wall Abutments and/or Wingwalls where Conventional Abutments and Wingwalls are Not Permitted (continued)

Include Conventional Abutment and Wingwall Design (Refer to Figure 3.3.4.6-6):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR	DECK
	UNIT							
EITHER LS	8010 0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX						
	LS							
		613 CY	CLASS 3 EXCAVATION	289	100	224		
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640	100	477		
		393 CY	CLASS A CEMENT CONCRETE	179	100	114		
		LS	STEEL BEAM TEST PILES, HP 12 X 53	2 @ 37'				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24' 1 @ 35'		
AND 14318	1002 0053	REINFORCEMENT BARS,EPOXY-COATED		2,270		186	1,000	28,107
	LB							
AND 1,672	1005 1103	STEEL BEAM BEARING PILES,HP 12 X 53		910		762		
	LF							
AND 75	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		42		33		
	EACH							
OR LS	8100 0002	STEEL BRIDGE STRUCTURE						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53		(---)		(---)		
	LF							

Figure 3.3.4.6-12 – Conventional Abutment and Wingwall Design

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR	DECK
	UNIT							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		(---)		(---)		
	EACH							
OR LS	8000 0003	PRESTRESSED CONCRETE BRIDGE STRUCTURE						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53		(---)		(---)		
	LF							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		(---)		(---)		
	EACH							

Figure 3.3.4.6-12 – Conventional Abutment and Wingwall Design (continued)

Does Not Include Conventional High Abutment and Wingwall Design (Refer to Figure 3.3.4.6-6):

Items shown on this table are incomplete for an actual report and are shown merely as an example of the proposed items layout.

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO.	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
	UNIT							
EITHER LS	8010 0001	BRIDGE STRUCTURE, AS DESIGNED, S-XXXXX						
	LS							
		613 CY	CLASS 3 EXCAVATION	289*	100	224*		
		1,217 CY	SELECTED BORROW EXCAVATION, STRUCTURE BACKFILL	640*	100	477*		
		393 CY	CLASS A CEMENT CONCRETE	179*	100	114*		
		LS	STEEL BEAM TEST PILES, HP 12 X 53	2 @ 37'				
		LS	STEEL BEAM TEST PILES, HP 12 X 53			1 @ 24' 1 @ 35'		
AND	1002 0053	REINFORCEMENT BARS, EPOXY-COATED		2,270		186	1,000	28,107
	LB							
AND	1005 1103	STEEL BEAM BEARING PILES,HP 12 X 53		910		762		
	LF							
AND 75	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53		42		33		
	EACH							
OR LS	8100 0005	STEEL BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES,HP 12 X 53		(---)		(---)		
	LF							

Figure 3.3.4.6-13 – Does Not Include Conventional High Abutment and Wingwall Design

TABULATION OF BRIDGE BID ITEMS & APPROXIMATE QUANTITIES								
QUANTITY	ITEM NO.	INFORMAL QUANTITY & UNIT	DESCRIPTION	NEAR ABUTMENT	PIER	FAR ABUTMENT	SUPERSTR.	DECK
	UNIT							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)			(---)		
	EACH							
OR LS	8000 0006	PRESTRESSED CONCRETE BRIDGE STRUCTURE (PREFABRICATED ABUTMENTS)						
	LS							
AND (---)	1005 1103	STEEL BEAM BEARING PILES, HP 12 X 53	(---)			(---)		
	LF							
AND (---)	1005 1153	STEEL BEAM PILE TIP REINFORCEMENT, HP 12 X 53	(---)			(---)		
	EACH							
LS	8640 0001	PREFABRICATED ABUTMENT OR WINGWALL, S-XXXXXP	(---) SF			(---) SF		
	LS							

(List Applicable Prefabricated Wall Systems on the Drawing)

*Adjust Quantities

Figure 3.3.4.6-13 – Does Not Include Conventional High Abutment and Wingwall Design (continued)

3.3.4.7 Requirements for Contractor Prepared Plans

The drawings shall include all details, dimensions, quantities, and cross-sections necessary to construct the wall. The plans shall be prepared to Department standards (see PP1.6) and shall include, but not be limited to, the following items:

- (a) A plan and elevation sheet or sheets for each wall, containing the following:
 - (1) An elevation view of the wall which shall indicate the elevation at the top of the wall at all horizontal and vertical break points, and at least every 50 ft. along the wall; elevations at the top of leveling pads and footings; the distance along the face of the wall to all steps in the footings and leveling pads; the designation of the type of panel or module; the length, size, and number of mesh or strips, and the distance along the face of the wall where changes in length of the mesh or strips occur; and the location of the original and final ground line
 - (2) A plan view of the wall which shall indicate the offset from the construction centerline to the face of the wall at all changes in horizontal alignment, limit of widest module, mesh, or strip, and the centerline of any drainage structure or drainage pipe behind or passing through or under the wall
 - (3) Any general notes required for constructing the wall
 - (4) All horizontal and vertical curve data affecting wall construction
 - (5) A listing of the summary of quantities provided on the elevation sheet of each wall for all items including incidental items
 - (6) A cross-section showing the limits of construction and, in fill sections, limits and extent of granular material placed above original ground
 - (7) Name of the material supplier
- (b) All details including reinforcing bar bending details
- (c) All details for foundations and leveling pads, including details for steps in the footings or leveling pads, as well as the factored bearing resistance and the maximum design bearing pressures
- (d) All details for panels and modules, showing all dimensions necessary to construct the element, all reinforcing steel in the element, identification of panels with epoxy-coated reinforcement, and the location of reinforcement attachment devices embedded in the panels
- (e) All details for construction of walls around drainage facilities, sign footings, and abutment piles
- (f) All details of the architectural treatment
- (g) All details for connections to barriers, copings, noise walls, and attached lighting
- (h) Detailed erection plan, particularly construction sequencing for the wall
- (i) Detailed computations for internal and external stability and life expectancy for reinforcement and hardware
- (j) A design summary in the tabulation format shown in Figures 3.3.4.7-1 and 3.3.4.7-2, as applicable.
- (k) When contractor designed alternate plans are submitted for proprietary walls, the S-number shown on the original design of the proprietary wall shall be suffixed by the letter P and appear on the alternate design plans.
- (l) The following note shall appear on sheet one of the alternate design plans (above title block):

For additional design information, core borings and other geotechnical information not shown on these plans, refer to the original design plans, (S-number of original design plans).

The plans shall be prepared and signed by a Professional Engineer registered in the Commonwealth of Pennsylvania.

The number of sets of design drawings and computations and who they shall be submitted to are given below:

- MSE Walls:

Two sets of design drawings and computations shall be submitted to the District Executive for review and approval.

- Precast Modular Walls:

Two sets of design drawings and computations shall be submitted to the District Executive for review and approval.

Any related shop drawings shall be reviewed according to Appendix B by the District and compared with the approved design drawings for the proprietary walls. Approval of the shop drawings and notifications shall be made by the District.

The computations shall be legible and shall include an explanation of any symbols and computer programs used in the design of overturning. As specified in PP3.3.4.5, bearing pressure beneath the wall footing or mechanically stabilized embankment, and estimated settlement shall be clearly indicated.

COUNTY: _____

LR/SR: _____

SECTION: _____

STATION: _____

S. NO.: _____

MSE WALL DESIGN SUMMARY

DATE: _____

(NOTE: FOR DEFINITION OF TERMS REFER TO A11.10)

WALL HEIGHT = H = _____.

REINFORCEMENT LENGTH = L = _____.

WALL NO. = _____.

REINF. LAYER NO.	DEPTH (ft.)	σ_v (ksf)	K	f*	Np	$\sigma_H = \gamma_p \sigma_v K$	Le (ft.)	SELECT STRIP/ MESH PANEL	$F_H = \sigma_H(A)$ (kips)	F_y STRIP = $F_{H/A'}$ (ksi)	F_y CONN. = $\Delta\Delta$ (ksi)	$P_n = \phi P_{fg}$ OR $P_n = \phi P_{fs}$	N = $\Delta\Delta\Delta$	P_i	REMARKS

Maximum Design Bearing Pressure = _____.

Factored Bearing Resistance = _____.

Maximum Design Eccentricity = _____.

Maximum Allowable Eccentricity = _____.

Design Horizontal Force = _____.

Maximum Design Horizontal Resistance = _____.

Calculated
Maximum
Settlement = _____. $\Delta\Delta$ = 85% of max. tie tension to a depth of factored $0.6H_1$,
increasing to 100% at the toe maximum of wall. $\Delta\Delta\Delta$ = No. of cross bars or strips in Maximum Design
Failure PlaneA = _____ A panel
No. of strips or mesh

A' = Area of strip or mesh

Figure 3.3.4.7-1 – Design Summary for Mechanically Stabilized Earth (MSE) Walls

COUNTY: _____
 LR/SR: _____
 SECTION: _____
 STATION: _____
 S. NO.: _____

MODULAR WALL DESIGN SUMMARY

DATE: _____

WALL HEIGHT = H = _____
 (Excluding Barrier)

WALL BATTER = _____.

WALL NO. _____

FRICITION ANGLE - BACKFILL = _____.

SURCHARGE: LEVEL _____, SLOPING _____, ABUTMENT _____

Course From Top	MODULE SIZE (in.)					WALL REINF. (Size/Spacing)			Design Eccentricity	REMARKS
	Height	Width	Length	Wall Thickness (in.)	Internal Pressure (ksi)	Hoop Bars	Vertical Bars	Epoxy- Coated Y/N		

Design Footing Size: Toe (Width x Thickness) _____.

Longit. Bars _____.

Transv. Bars _____.

Provide Footing Size: Toe (Width x Thickness) _____.

Calculated Maximum Settlement = _____.

Maximum Design Horizontal Force = _____.

Maximum Design Horizontal Resistance = _____.

Heel (Width x Thickness) _____.

Longit. Bars _____.

Transv. Bars _____.

Heel (Width x Thickness) _____.

Maximum Design Bearing Pressure = _____.

Factored Bearing Resistance = _____.

Figure 3.3.4.7-2 – Design Summary for Modular Walls

3.4 BRIDGE PROTECTIVE SYSTEMS

3.4.1 Overlays

Generally, overlays should not be provided for new construction (for additional guidance, see D9.11.3P). However, if it becomes necessary to provide an overlay to correct for a poor riding surface or for other reasons, a 1 1/2 in. thick latex-modified concrete overlay is preferred.

For treatment of existing decks, refer to PP5.5.2.3.

3.4.2 Protective Coatings for Concrete Surfaces

(a) Superstructure

For bridge decks that are poured and opened to traffic between September 1 and March 1, a general note shall be added to the design plans specifying the application of a concrete sealant in accordance with Publication 408, Section 1001.3(k)6.

(b) Substructure

Concrete sealants shall be specified for substructure units in accordance with D5.4.3.6P(b).

3.4.3 Cathodic Protection

Cathodic protection is considered an effective means of stopping and preventing corrosion of reinforcement bars in concrete. Refer to PP5.5.2.7.

3.4.4 Protective Coatings for Steel

Generally, all new steel bridges shall be painted with an inorganic zinc-rich painting system as specified in Publication 408 for painting fabricated structural steel. Consideration may also be given to galvanizing or metallizing of steel members. When considering a galvanizing protection system, fabrication issues such as galvanizing tank sizes and lifting capacities of galvanizing facilities need to be evaluated. Galvanizers listed in Bulletin 15 should be consulted as to their capabilities and the feasibility of the process for the specific steel member(s) being considered. For metallizing of steel members see D6.7.3.

Certain rural locations in the state may have low rates of air pollution and, therefore, a steel superstructure would not be exposed to spray generated from deicing chemicals. For such a location, the use of unpainted weathering steel may be considered, provided there is little potential for industrial development or significant increases in traffic in the area (see D6.4.1).

For epoxy-coated rebar see D5.4.3.6P.

3.4.5 Vitriified-Clay Liner Plates

The use of vitriified-clay liner plates shall be specified for any concrete structure that is located in a stream having a hydrogen ion concentration (pH) of 5.0 or less as determined by a laboratory analysis of a representative sample of water taken from the stream during a period of normal water level. The liner plates shall be placed on the face of the concrete from approximately 18 in. below the streambed to approximately 18 in. above normal water elevation. Grade SA sewer bricks meeting the requirements of ASTM C 32-09 may be used for curved portions of the structure when the use of vitriified-clay liner plates is not practical.

The samples of water should be obtained when a representative degree of acidity is present. In recommending the use of vitriified-clay liner plates, the worthiness of the sample should be taken into consideration. A pH factor of 7.0 indicates that the water is neutral; a pH factor less than 7.0 shows the degree of acidity of the water; and a pH factor greater than 7.0 shows the degree of alkalinity.

3.4.6 Other Protective Systems

Other protective systems which are applicable to specific items of construction (e.g., piles, buried structures, MSE walls, and permanently anchored walls) may be found in the sections of the Manual that cover the respective items.

3.5 UTILITIES

3.5.1 Service Utilities

The design and review of bridge attachments and responsibility to assure compliance with attachment and license requirements are basic functions of District Bridge Units.

The Bridge Design and Technology Division of the Bureau of Project Delivery, Central Office, will provide additional guidance if necessary for the particular bridge attachment.

District Utility Units will act as the liaison between utility companies and District Bridge Units.

Refer to Design Manual, Part 5, Utility Relocation, Publication 16M, Chapter 1.3.B.6, for Utility Occupancy of Highways and Bridges, for general guidelines coordination procedure and guidelines for accommodation of utilities on structures.

3.6 MISCELLANEOUS

3.6.1 Bridge Lighting and Navigational Lighting

For lighting of bridges, see Design Manual, Part 1C, Chapter 4. Bridge-mounted highway lighting shall be avoided wherever possible. The designer shall investigate the possibility of mounting the lighting on an extended pier cap. If bridge-mounted lighting cannot be avoided, it shall be located as close to a pier as is practical. Lighting poles shall be installed upon completion of the entire superstructure to prevent damage due to traffic on half-width of deck slab. The designer shall insure that vibration of the lighting pole at the luminare is not exceeded by the vibration induced by traffic on the as-designed bridge by providing computational proof.

3.6.2 Energy-Absorbing Devices

For policy on impact attenuators, refer to Design Manual, Part 2, Chapter 12.

3.6.3 Sign Structures

For implementation of the design, design review and fabrication control of sign structures, see Design Manual Part 1C, Chapter 4.

Standard Drawings BD-641M, BD-642M, BD-643M, BD-644M and BD-645M shall be followed for design of sign structures. Standard Drawings BC-741M, BC-742M, BC-743M, BC-744M and BC-745M shall be followed for fabrication and construction of sign structures. For design specifications, refer to AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, 4th edition, 2001 including Interims through 2006 and AASHTO/AWS D1.5M/D1.5 – Bridge Welding Code (refer to AWS D1.1 for welding not covered in D1.5).

Refer to ITS-1003M (Pub. 647M) for preparation of DMS structure layouts and contract drawings.

Selection of the type of sign structure should consider initial construction cost, type of sign (aluminum or VMS), foundation sizes, and the ability to galvanize the structure. Recommended sign structure type is 4-post 4-chord to standardize the general appearance of the sign structures within the state as well as with neighboring states.

The “Design Sign Area” will be set by the Traffic Engineer based on the structure location, rural/urban. The actual sign area may be increased up to 25% for rural highways and 50% for urban highways to establish the “design sign area” to be used for the design of the sign structure. Any increase in sign area above 25% (rural) or 50% (urban) of the actual sign area requires approval by the Chief Bridge Engineer.

It is preferable to galvanize a field section of a sign structure in a single dip as opposed to double dipping the field sections. Double dipping of field sections has caused component failures during the galvanizing process. To determine if a field section can be galvanized in a single dip, the designer should discuss with local galvanizers listed in Publication 35 Bulletin 15 the depth of a field section (out-to-out of the chord splice plates), the weight of a field section and length of a field section. If the structure cannot be galvanized in a single dip by a local galvanizer, then a different sign structure type should be evaluated, e.g. for a trichord structure which cannot be galvanized in a single dip, then a 4 chord structure should be evaluated.

All sign supports located within the clear zone must be shielded with a crashworthy barrier. If a barrier is required or used, the sign bridge shall be located just beyond the design deflection distance of the barrier to minimize the required span length.

For caisson foundation, core borings are recommended unless geology is known, such as roadway borings show consistency.

Sign structure shop drawings are to be reviewed and accepted at the District level.

3.6.3.1 Bridge-Mounted Signs

Signs shall be aligned parallel to the bridge if the skew angle is 80° or more. Otherwise, the signs shall be perpendicular to the traveling lanes underneath. For horizontally curved roadway for which the sign is placed on a bridge, the sign shall be perpendicular to the chord joining a point 350 ft. away from the sign and the sign location. This value may be reduced on low speed roads. The bottom of the luminaries or sign shall be 3 in. above the bottom of the fascia girder. The sign support brackets may be attached to barrier and/or girders using standard, approved details. Lock nuts or lock washers shall be used to compensate for bridge vibration effect.

3.6.3.2 Bridge-Mounted Sign Structure (Overhead Signs)

Bridge-mounted sign structures shall be avoided wherever possible. If they cannot be avoided, they should be located as close to a pier support as is practical. The affected bridge components shall be designed to carry the additional loads due to the weight of the sign structures and wind. All sign supports located within the clear zone must be shielded with a crashworthy barrier. If a barrier is required or used, the sign bridge shall be located just beyond the design deflection distance of the barrier to minimize the required span length.

3.6.4 Sound Barriers Walls

3.6.4.1 General

- (a) For acoustic requirements of barriers, wall sizing and location, see Design Manual, Part 1C, Chapter 4.
- (b) Wall heights must equal or exceed the acoustical profile.
- (c) Design sound barrier walls in accordance with the Working Stress Design (WSD) method.
- (d) Construct sound barrier walls using precast concrete or steel posts and precast concrete panels in accordance with the Standard Drawings. Alternate wall types using steel, concrete, timber, masonry, plastic, or any other material must be approved by the Department prior to bidding. The use of aluminum is not permitted.
- (e) Any proprietary sound barrier wall system used on Department projects must be approved through the Department evaluation and approval process, as specified in the Master Policy Statement (MPS) 418, prior to use during the design or construction phase (see PP1.14, Systems Approval). Only accepted sound barrier systems (refer to Publication 35, Bulletin 15 for approved suppliers) shall be used.
- (f) Refer to Standard Drawings BC-776M, BC-777M, BC-778M, BC-779M, and BC-780M for fabrication and construction of sound barrier walls.
- (g) Refer to Standard Drawings BD-676M, BD-677M, BD-678M, BD-679M, and BD-680M for design of sound barriers walls.
- (h) Structure-related environmental commitments shall be carefully considered and justified. Where practical, avoid sound barriers or minimize their size by other mitigative measures including earth berms. If possible, refrain from making commitment on material type, i.e., steel, concrete, timber, masonry, etc., until after the final design stage, when all reasonable options can be considered by the designer for a structurally and economical sound barrier. Where it is necessary to deviate from standards, justifications and special approvals must be well documented.
- (i) Provide fire hydrant openings or other highway access as required. Provide additional reinforcement around openings in accordance with the Standard Drawings.
- (j) Provide access doors in the wall if required by local fire departments, for inspection, or if directed by the District Bridge Engineer, in accordance with the Standard Drawings.
- (k) Provide a constant post spacing for the entire length of wall. The constant post spacing may be interrupted to miss drainage pipes, utilities, expansion joints, construction joints, and/or any other physical features. Variations must be accepted by the District Bridge Engineer.

- (l) Sound Absorptive Panels: Indicate if sound absorptive panels are required to reduce the reflective sound when sound barriers are on both sides of the highway and are spaced closer than 100'-0" and if required by the noise study. If a sound absorptive material is required, only use a system that is Pre-Approved by the Department.
- (m) Provide construction and fabrication tolerances in accordance with Publication 408, Section 1086.
- (n) Provide S-Number in accordance with PP1.6.2.5.
- (o) Plan presentation shall be in accordance with PP1.6 and PP3.6.4.12.
- (p) Provide uncoated reinforcement bars in the footings and caissons, unless otherwise specified.
- (q) Provide epoxy coated or galvanized reinforcement bars in the precast concrete posts, precast concrete panels, pedestals, and raised panel seats where the wall is within 14'-0" of the edge of travel lane. Epoxy coated or galvanized reinforcement bars may be required if future widening is anticipated.

3.6.4.2 Wall Types

(a) Ground Mounted Sound Barrier Walls

(1) General

- Provide a minimum panel ground embedment of 6 inch, unless otherwise specified.
- Grade ground to drain surface water away from wall. Provide slopes so water will not pond at or near the sound barrier.
- Ground mounted sound barrier walls must be protected by concrete barriers, unless the wall is located beyond the horizontal clear zone or if the bottom of the panels are a minimum of 5'-0" above the edge of pavement.
- Earth berms are permitted to be used in conjunction with Linear and Offset Walls. The top of the earth berm must be wide enough to permit construction of the wall.

(2) Linear Walls (Post and Panel)

- Minimum Wall Height (Top of wall to bottom of bottom panel) = 4'-0"
- Maximum Wall Height (Top of wall to bottom of bottom panel) = 30'-0"
- Maximum Post Spacing = 20'-0"
- Precast concrete posts with precast concrete panels (Refer to Standard Drawings BC-776M, BC-777M, BD-676M, and BD-677M)
- Steel posts with precast concrete panels (Refer to Standard Drawings BC-776M, BC-778M, BD-676M, and BD-678M)
- Timber posts with timber panels
- Masonry block
- or any other pre-approved wall system.

(3) Offset Walls

- Offset walls are defined as freestanding walls undulating to create a "fan" type appearance supported on a cast-in-place spread footing.

- Minimum Wall Height (Top of wall to top of footing) = 6'-0"
- Maximum Wall Height (Top of wall to top of footing) = 30'-0"
- Provide a minimum panel ground embedment of 2'-0".
- Refer to Standard Drawing BC-780M and BD-680M for additional information.

(4) Earth Berms

- Earth berms are defined as berms constructed from natural earthen materials to act as a "natural" sound barrier. These types of barriers are typically constructed with surplus materials available from the project or materials transported from an off-site location.
- Construct earth berms in accordance with the Department's criteria.
- The following factors shall be considered when selecting earth berms:
 - Right-of-Way requirements
 - Visual implications
 - Maintenance and accessibility
 - Drainage implications

(b) Structure Mounted Sound Barrier Walls

(1) General

- Precast concrete posts are not permitted for structure mounted sound barrier walls, provide steel posts.
- Maximum Post Spacing = 12'-0" unless otherwise specified.
- Provide lock nuts or lock washers due to structure vibrations in accordance with the Standard Drawings.
- Steel cables are required in all structure mounted precast concrete panels unless both sides of the panel are located beyond the horizontal clear zone or if the bottom of the panels are a minimum of 5'-0" above the edge of pavement. Steel cables are always required in the bridge mounted precast concrete panels. (Refer to BC-779 for steel cable details.)
- For non-concrete panels, alternate means of positive connection must be provided if used in applications requiring cables, as stated above. The connections must be approved by the Chief Bridge Engineer prior to bidding.
- Mount sound barriers to structures in accordance with Standard Drawing BC-779M and BD-679M.
- Structure mounted sound barrier walls shall be designed and detailed to maintain bridge inspectability. For special conditions, the inspectability shall be determined by the Chief Bridge Engineer.
- Slip forming is not permitted for concrete traffic barriers when sound barriers are required.

(2) Sound Barrier Mounted on Traffic Barrier on Bridges (Steel Posts with Precast Concrete Panels)

- Minimum Wall Height (Top of wall to top of barrier) = 4'-0"
- Maximum Wall Height (Top of wall to top of barrier) = 10'-0"
- Minimum Traffic Barrier Height = 3'-6"

- Provide light weight precast concrete panels on bridges when required in accordance with the requirements of Standard Drawing BD-601M.
 - Stacked panels are not permitted on bridges.
 - Design the traffic barrier, deck slab, and beams/girders considering the additional loads due to the sound barrier wall.
 - Make provisions to allow for bridge movement in the design of bridge mounted sound barriers. (Refer to BD-679M for details)
 - Provide expansion panels over bridge expansion joints. (Refer to BD-679M for details.)
- (3) Sound Barrier Mounted on Traffic Barrier on Retaining Walls or Moment Slabs (Steel Posts with Precast Concrete Panels)
- Minimum Wall Height (Top of wall to top of barrier) = 4'-0"
 - Maximum Wall Height (Top of wall to top of barrier) = 15'-0"
 - Minimum Traffic Barrier Height = 3'-6"
 - Design the traffic barrier, retaining wall, and/or moment slab considering the additional loads due to the sound barrier wall.
 - Provide expansion panels when shoulder relief joint is required in the moment slab. (Refer to BD-679M for details.)
- (4) Sound Barrier Mounted on Retaining Wall (Steel Posts with Precast Concrete Panels)
- Minimum Wall Height (Top of wall to top of retaining wall) = 4'-0"
 - Maximum Wall Height (Top of wall to top of retaining wall) = 20'-0"
 - Design the retaining wall considering the additional loads due to the sound barrier wall.

3.6.4.3 Geometry and Layout

- (a) All sound barrier walls located within the clear zone must be protected with a concrete barrier in accordance with the requirements of Design Manual, Part 2, Chapter 12. The positive protection from vehicles preclude the need for designing the sound barrier wall for a traffic impact load.
- (b) Generally the alignment of the sound barrier shall be constructed at a continuously uniform distance from the roadway.
- (c) Minimum horizontal distances (sight and stopping) shall be considered when determining sound barrier alignment.
- (d) Avoid abrupt changes in the horizontal and vertical alignment of the sound barrier.
- (e) If sound barrier is located along a curved roadway alignment, the alignment of the sound barrier is permitted to be broken up into chorded sections in order to simplify the layout of the sound barrier.
- (f) Provide angled or corner posts when required. Refer to BC-777M and BC-778M for angle limitations.
- (g) Locate sound barriers to avoid conflicts with utilities, drainage pipes, and/or any other physical feature.
- (h) If the sound barrier height exceeds 15'-0" consider using a sound barrier in combination with an earth berm to reduce the structure height of the sound barrier.

- (i) Sound barriers will obstruct light as well as sound. Special consideration shall be given to possible roadway icing and other induced environmental conditions caused by the placement of the wall.
- (j) Access to the residential side of the sound barrier shall be considered for inspection and maintenance.
- (k) Ends of Sound Barriers Walls
 - Ends of the sound barrier should be reduced in height (top of barrier to ground line) from their acoustically required height to a height of approximately 5'-0" (to match Right-of-Way fence height). This may be accomplished by using sloped end panels or by increasing the sound barrier length and sloping or stepping the panels to create a more aesthetically pleasing sound barrier. Options are to be discussed with the Department and accepted by the District Bridge Engineer.
 - Ends of the sound barrier could also be buried into existing or proposed slopes if the topography permits.
 - If using precast concrete posts, provide end posts in accordance with BC-777M and BD-677M.
- (l) Overlapping Sound Barriers Walls
 - Sound barriers which overlap each other are usually constructed to allow access gaps for maintenance, inspection, or safety purposes. The general rule-of-thumb is that the ratio between the overlap distance and gap width shall be at least 4:1 to ensure negligible degradation of sound barrier performance. The location of the access gaps, if required, shall be coordinated with the Department.
- (m) Structure Mounted to Ground Mounted Connection
 - Refer to BD-679M for transition details from a structure mounted sound barrier wall to a ground mounted sound barrier wall.

3.6.4.4 Public Involvement and Aesthetics

- (a) Refer to PennDOT Publication No. 24, Project Level Highway Traffic Noise Handbook.
- (b) Refer to PennDOT Publication No. 21, Making Sound Decisions about Highway Noise Abatement.
- (c) Determine aesthetics considering the surrounding landscape and local architectural features. Aesthetics shall also be coordinated and approved by the District Environmental Manager.
- (d) When meeting with the public only present wall types which are in accordance with the Standard Drawings or Pre-Approved by the Department as alternates.
- (1) Consider and permit all feasible alternates. The Sound Barrier Standard Drawings are to be used unless restricted by the following requirements:
 - Acoustical profile requirements
 - Sound absorptive panels requirements
 - Architectural surface treatments requirements
 - Wall alignment
 - or any other requirement

The above restrictions must be discussed and accepted by the Department prior to public involvement.

- (2) If only one proprietary type is feasible it shall be specified, provided that justification is given for this selection (economic, noise abatement) and that approval has been obtained from the Chief Bridge Engineer prior to bidding.

(e) Architectural Surface Treatments (texture) and Color:

- (1) The Department will decide the color and texture on the highway side of the proposed sound barrier wall unless there is third party funding involved.
- (2) The public will be presented options for the color and texture on the residential side of the proposed sound barrier. The final decision shall be determined by the Project Team based on the opinions of the public.
- (3) The color and texture choices shall be limited to prevent numerous different colors and textures on a given project. The sound barrier walls shall be consistent on a wall-to-wall basis, on the project, to simplify construction and to be economical.
- (4) In some urban areas, sound barriers may be subjected to graffiti on their surfaces. In these locations, the surface texture selected shall be such that it is difficult to place the graffiti or such that the graffiti is easily removed. Sound barriers with rough textures and dark colors tend to discourage graffiti.
- (5) Color on concrete surfaces may be obtained by using penetrating concrete stain and/or integral concrete stain.
- (6) Color on steel surfaces shall be obtained by painting after galvanizing.
- (7) Avoid using form liner finishes on both sides of the precast concrete panels. A form liner finish, on one side of the panel, along with a stamped finish, on the other side of the panel is permitted. Preferred option is to only use a form liner finish on one side.
- (8) Raked or broom finishes are only permitted to be on one side of the panel.
- (9) Stamped finishes may be permitted if accepted by the District Bridge Engineer.
- (10) Form liner finishes are not recommended on the precast concrete posts.
- (11) The architectural surface treatment thickness, on each side of precast concrete panel, is permitted to vary from 0 to 1½ inch, but the total average architectural surface treatment thickness, on both sides of the precast concrete panel, must not be greater than 1½ inch. Thicker architectural surface treatments may be permitted if accepted by the District Bridge Engineer and accounted for in the design of the precast concrete panel.

3.6.4.5 Design Specifications and Design Loads

- (a) AASHTO Guide Specifications for Structural Design of Sound Barriers, 1989, and Interim Specifications 1992 and 2002.

Design loads and loading combinations of dead load, lateral earth pressure, live load surcharge, wind load, seismic load, ice load, and traffic impact load shall be in accordance with this Guide Specifications and as modified herein.

Modify the following sections as follows:

SECTION 2 - LOADS**1-2.1 Applied Loads****1-2.1.1 Dead Load**

Add the following:

Unit Weight of Normal Concrete = 150 pounds per cubic foot

Unit Weight of Light Weight Concrete = 115 pounds per cubic foot

Unit Weight of Soil above top of Footing/Drilled Caisson = 100 pounds per cubic foot

1-2.1.2 Wind Load

Delete this section and replace with the following:

The design wind pressure for Ground Mounted and Structure Mounted Sound Barrier Walls shall be as indicated below.

The wind load includes a gust factor of 1.3 and a drag factor of 1.2 and is based on a maximum 50-year Mean Wind Velocity at 30'-0" above the ground surface of 80 mph.

Height Zone is defined as the distance from the adjoining (adjacent) average ground line (streambed or lower roadway level for grade separation structures) to the centroid of the sound barrier.

(a) Ground Mounted Sound Barrier Walls (including Offset Walls)

Height Zone (ft.)	Wind Pressure (psf)
0 - 14.0	20
Over 14.0	28

(b) Structure Mounted Sound Barrier Walls

Design structure mounted sound barrier walls for a wind pressure equal to 37 psf.

1-2.1.5 Traffic Loads

Add the following:

Sound barrier walls are to be designed and detailed so they will not be impacted by traffic loads.

1-2.1.7 Ice & Snow Loads

Delete this section and replace with the following:

Ice load shall be based on a pressure on the wall of 0.003 ksf (3 psf) applied at only one face of the wall.

1-2.2 Load Combinations

1-2.2.1 Working Stress Design (WSD)

Use the following Group Loads and Allowable Overstresses:

Group I: $D + E + SC$ 100%
 Group II: $D + W + E + SC$ 100%
 Group III: $D + EQD + E$ 133%
 Group IV: $D + W + E + I$ 100%

1-2.2.2 Load Factor Design (LFD)

Delete this Section.

1-2.2.3 Strength Reduction Factors, (ϕ)

Delete this Section.

SECTION 3 - CONCRETE

Use fully reinforced sections only, including all foundation components. In case of conflict between this Guide Specification and Department criteria in DM-4, the Department criteria will govern.

SECTION 8 – FOUNDATION DESIGN

1-8.2 Spread Footings

Delete the Minimum Factors of Safety for Overturning and Sliding and replace with the following:

Minimum Factors of Safety for Overturning for Footings supported on Soil:

- Group I = 2.0
- Group II = 2.0
- Group III = 1.50
- Group IV = 2.0

Minimum Factors of Safety for Overturning for Footings supported on or embedded in Rock:

- Group I = 1.50
- Group II = 1.50
- Group III = 1.125
- Group IV = 1.50

Minimum Factors of Safety for Sliding:

- Group I = 1.50
- Group II = 1.50
- Group III = 1.125
- Group IV = 1.50

Add the following:

Minimum Factor of Safety for Bearing Capacity for Footing supported on soil and rock:

- Group I = 3.0
- Group II = 3.0
- Group III = 1.5
- Group IV = 3.0

- (b) AASHTO Standard Specifications for Highway Bridges, 17th Edition.

Modify the following sections as follows:

SECTION 8 - REINFORCED CONCRETE

8.1 APPLICATION

8.1.1 General

Add the following:

Design sound barrier walls in accordance with the Working Stress Design (WSD) Method.

8.1.2 Notations

Revise notation of f'_c as follows:

f'_c = structural design strength, psi

8.2 CONCRETE

Replace A8.2 as follows:

Minimum mix design compressive strength (psi) shall be in accordance with Section 704.1(b) of Publication 408.

The following classes of cement concrete with the corresponding f'_c are to be used for structural designs:

Class of Cement Concrete	f'_c Structural Design Strength
AA, Modified	5,000 psi
A	3,000 psi
C	2,000 psi

The use of different classes of cement concrete shall be as follows:

Class AA Cement Concrete, Modified

- Precast Concrete Panels
- Precast Concrete Posts

Class A Cement Concrete

- Pedestals
- Raise Panel Seats
- Footings
- Caissons

Class C Cement Concrete

- below bottom of footings when specified

Show the structural design strength (f'_c) of the concrete for each part of the structure on the plans.

8.3 REINFORCEMENT

Supplement A8.3 as follows:

Provide Grade 60 [$f_s = 24,000$ psi] deformed reinforcement bars. Do not weld reinforcement bars.

Provide Grade 65 [$f_s = 24,000$ psi] plain welded wire fabric in the precast concrete panels.

Provide Grade 70 [$f_s = 24,000$ psi] deformed welded wire fabric in the precast concrete posts.

8.3.5

Delete A8.3.5

8.5 EXPANSION AND CONTRACTION

8.5.3

Add the following:

The coefficient of thermal expansion and contraction for light weight concrete shall be taken as $5.0 \times 10^{-6}/^{\circ}\text{F}$.

8.6 STIFFNESS

Supplement A8.6 as follows:

8.6.3P Moment of Inertia

The value of the moment of inertia for the computation of flexural stiffness of slabs, beams, columns, etc., shall be based on gross concrete section with the effect of reinforcement neglected.

8.19 LIMITS FOR SHEAR REINFORCEMENT

8.19.1 Minimum Shear Reinforcement

8.19.1.3

Minimum Shear reinforcement requirements are not to be waived on any projects in Pennsylvania.

8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT

8.20.1

Supplement A8.20.1 as follows:

The minimum temperature steel reinforcement of $0.125 \text{ in}^2 / \text{foot}$ must always be met when the member is subject to temperature variation. Any member subject to loading or stress shall have minimum steel reinforcement of No. 4 bars at 12 inches or No. 5 bars at 18 inches. For ties in reinforced concrete I-posts for sound walls, the minimum steel reinforcing shall be No. 3 bars at 9 inches. The temperature reinforcement requirements are also met with this steel provided the spacing does not exceed three times the member thickness. Any exception to these criteria must be approved by the Chief Bridge Engineer.

8.20.3P Minimum Temperature Reinforcement

The requirements of AASHTO Article 8.20 must be met when any member is subject to temperature variation. Parts permanently embedded below 3'-0" in the ground or below frost level may be considered not subject to temperature variation.

8.22 PROTECTION AGAINST CORROSION

8.22.1

The following shall replace A8.22.1.

The following minimum concrete cover shall be provided for reinforcement:

- Concrete cast against and permanently exposed to earth: 4 in.
- Concrete exposed to earth: 3 in.
(Except 2 in. may be used in raised panel seats)
- Concrete exposed to weather: 2 in.
(Except 3 in. for ties in drilled caissons and pedestals)
- Precast Concrete Panels: $1\frac{1}{2}$ in.
- Precast Concrete Posts: $1\frac{3}{4}$ in.

SECTION 10 - STRUCTURAL STEEL

10.1 APPLICATION

10.1.2P Design Method

Design sound barrier walls in accordance with the Working Stress Design (WSD) Method.

10.8 MINIMUM THICKNESS OF METAL

10.8.1 Structural Steel

Replace A10.8.1 with the following:

The web thickness of rolled beams shall not be less than 0.23 inch. The minimum base plate thickness shall be 3/4 inch.

10.24 FASTENERS (Rivets and Bolts)

10.24.5 Spacing of Fasteners

10.24.5.3

Replace A10.24.5.3 with the following:

When oversize or slotted holes are used, the minimum clear distance between edges of adjacent bolt holes in the direction of force and transverse to the direction of the force shall not be less than twice the diameter of the bolt.

10.24.7 Edge Distance of Fasteners

10.24.7.3

Replace A10.24.7.3. with the following:

When oversize or slotted holes are used, the clear distance between edges of hole and edges of members shall not be less than the diameter of the bolt

- (c) AASHTO Standard Specifications for Structural Supports of Highway Signs, Luminaries and Traffic Signals, 4th Edition, 2001 including Interims through 2006.

3.6.4.6 Precast Concrete Panels

- (a) Provide Class AA Cement Concrete, Modified in the precast concrete panels.
- (b) Provide normal weight concrete for the ground mounted sound barrier panels.
- (c) Provide normal weight concrete panels for the structure mounted panels for the barriers mounted on retaining walls and moment slabs.
- (d) Provide either normal weight concrete or light weight concrete panels for the structure mounted panels for the barriers mounted bridges. Refer to BD-601M for requirements.
- (e) Panels shall be designed as simply supported beams using a 1'-0" strip width.

- (f) Minimum Structural Panel Thickness:
- (1) Panels for Linear Ground Mounted and Structure Mounted Walls = 4 1/2 inch
(Note: Panels in the Standard Drawings are designed using a 5 inch structural thickness.)
 - (2) Panels for Offset Ground Mounted Walls = 10 inch
- (g) Architectural Surface Treatment Thickness: Refer to PP3.6.4.4, item (e)(11).
- (h) Wind Load for Panel Design:
- (1) Ground Mounted Sound Barriers Panels: Design all panels using a wind pressure equal to 28 psf.
 - (2) Structure Mounted Sound Barrier Panels: Design all panels using a wind pressure equal to 37 psf.
- (i) Design panels to include the additional weight due to the architectural surface treatment thicknesses. Architectural surface treatment thicknesses are not permitted to be considered as a load carrying element. Stresses shall be calculated using only the structural panel thickness.
- (j) The minimum horizontal and vertical reinforcement shall not be less than 0.001 times the gross area of the panel. The gross area shall include the structural panel thickness plus the average architectural surface treatment thickness.
- (k) In addition to the group loads indicated in PP3.6.4.5, item (a), design panels for stresses due to stripping, handling, erection, and transportation in accordance with the Precast/Prestressed Concrete Institute (PCI) Design Handbook, Precast and Prestressed Concrete, 5th Edition, 1999, Chapter 5.
- (1) Design panels both horizontally and vertically using the following equivalent static load multipliers in accordance with Table 5.2.1:
 - Stripping caused by Form Suction and Impact = 1.50
The 1.50 factor is for the condition when the panels are cast horizontally and lifted/stripped from the form. If panels are cast using a tilt-up table, the 1.50 factor may not be applicable and a reduced factor may be more appropriate. The reduced factor must be accepted by the Chief Bridge Engineer.
 - Yard Handling = 1.20
 - Erection = 1.20
 - Transportation = 1.50

The equivalent static load multiplier is applied to the weight of the panel and used as an equivalent static service load.
 - (2) For stripping and yard handling provide a minimum concrete compressive strength of 80% of the 28-day minimum design compressive strength.
 - (3) Design panel thickness and reinforcement for stripping and lifting using a two-point pickup or four point pickup in accordance with Figure 5.2.4. Place lifting inserts at the locations indicated in Figure 5.2.4. Lifting inserts are permitted to be on the top and one side of the panel as required for stripping and erection.
 - (4) In accordance with Section 5.2.4.1, the structural panel thickness shall be adequate such that the flexure tensile stress in the concrete using the uncracked gross section, while neglecting the reinforcement, is less than the Modulus of Rupture reduced by a safety factor of 1.50.
- (l) Maximum permitted panel deflection equals panel length in inches divided by 360.
- (m) Welded wire fabric is the preferred type of reinforcement in the precast concrete panel. #4 reinforcement bars may be substituted for welded wire fabric with an equivalent area at no additional cost to the Department.

- (n) Provide perimeter reinforcement in the panel in accordance with the Standard Drawings. Minimum bar size shall be #4 for linear and structure mounted panels. Minimum bar size shall be #5 for offset panels.
- (o) Minimize the number of horizontal panel joints and provide uniform steps. If steps are required, the elevation difference between adjacent panels is not permitted to be less than 6 inches or greater than 2'-0". Stacked panels are not permitted for bridge mounted barriers.
- (p) Provide a 2'-0" maximum step between adjoining panels when stepping panels is required.
- (q) Indicate if the top of panels are stepped or sloped. Sloped panels are preferred.
- (r) Install panels truly vertical.

3.6.4.7 Posts

(a) General

- (1) Posts shall be designed as vertical cantilever beams.
- (2) Maximum permitted post deflection equals post height (cantilever length) in inches divided by 360. The effects of rotation and deflection at the top of the drilled caisson shall be ignored and only the relative displacement between the top of the caisson and top of post due to the applied loads shall be considered when calculating the deflections.
- (3) Install posts truly vertical.

(b) Precast Concrete Post and Connections

(1) General

- Provide Class AA Cement Concrete, Modified in the precast concrete posts.
- Provide normal weight concrete for the ground mounted sound barrier posts.
- The minimum vertical post reinforcement ratio shall be in accordance with the following equation:

$$\rho_{\min} = \left[10 + \frac{(l/y_t)}{(bd^2)} \right] \left[\frac{(l/y_t)}{(bd^2)} \right] \frac{\sqrt{f_c}}{f_y}$$

(2) Precast Concrete Post with Steel Base Plate and Anchor Bolts supported on Drilled Caissons or Spread Footings

- Provide epoxy coated or galvanized vertical post reinforcement bars.
- Provide threads on one end of the bar. Provide either normal threads on bar or provide upset threads. Specify type of bar to be used on the plans.
 - Normal Threads: Specify bar size and cut threads at one end.
 - Minimum vertical post reinforcement bar size shall be #5.
 - Upset Threads: Oversize bar with specially forged end for the length of threads.
 - Minimum vertical post reinforcement bar size shall be #4.
- Design threaded reinforcement bars using the net tensile stress area of the threaded bar.
- Galvanize the steel base plate, anchor bolts and nuts.

- Paint exposed galvanized components to match wall color.
 - Refer to PP3.6.4.8 for base plate design.
 - Refer to PP3.6.4.9 for anchor bolt design.
- (3) Precast Concrete Post Embedded in Drilled Caisson or Spread Footing (with or without pedestal)
- Provide plain, galvanized, or epoxy coated vertical post reinforcement bars as required.
 - Minimum vertical post reinforcement bar size shall be #4.
 - Post embedded in Drilled Caisson
 - The precast concrete post must be embedded to a depth where the caisson reinforcement is fully developed.
 - Vertical post reinforcement bars must extend a development length beyond the end of the post into the drilled caisson, except when the post is embedded to the bottom of the drilled caisson.
 - Post embedded in Spread Footing
 - The precast concrete post must be embedded a minimum of 6 inches into the spread footing.
 - Provide 90 degree hooks on the vertical post reinforcement bars. The bar must extend a hooked development length beyond the end of the post. The hook length shall be based on a standard 90 degree hook.
- (4) Precast Concrete Angled and Corner Post Embedded in Drilled Caisson or Spread Footing (with or without pedestal)
- Provide angle posts when the intersecting angle between adjacent panels is greater than 138 degrees and less than 162 degrees.
 - Provide corner posts when the intersecting angle between adjacent panels is greater than 78 degrees and less than 102 degrees.
 - Design Conditions:
 - Design Condition 1:
Design post for a wind direction normal to the post with an effective width equal to the post spacing multiplied by the sine of the one-half the angle between the centerline of the panels. Design must investigate wind from both directions. The depth of the compression block must be verified so it does not extend beyond the flange of the post and that the tensile reinforcement is adequate. The post must also be designed for flexure, shear, deflection, minimum shear reinforcement, minimum flexural reinforcement, and flange bending stresses in accordance with the requirements of AASHTO and Design Manual, Part 4.
 - Design Condition 2:
Design post for a wind direction normal to the panel with an effective width equal to one-half the post spacing applied to only one side of the post. Design must investigate wind from both directions and the combined torsion and shear capacity of the post must be verified. Checked torsion in accordance with ACI 318-99, Section 11.6.1. The post must also be designed for flexure, shear, deflection, minimum shear reinforcement, minimum flexural reinforcement, and flange bending stresses in accordance with the requirements of AASHTO and Design Manual, Part 4.
 - For additional information refer to PP3.6.4.7, item (b)(3).

(c) Steel Posts and Connections

(1) General

- Provide structural steel conforming to AASHTO M270M Grade 36 [ASTM A709, Grade 36], unless otherwise noted.
- The structural steel designed in the Sound Barrier Standard Drawings use AASHTO M270M Grade 36. Grade 50 is permitted to be used as an alternate.
- Galvanize and paint the steel posts, base plates, anchor bolts and nuts.
- Weathering Steel (A588) is not permitted.
- Allowable Bending Stress shall be in accordance with AASHTO Standard Specifications for Highway Bridges, 17th Edition, Section 10.32.1.

(2) Steel Post with Steel Base Plate and Anchor Bolts supported on Drilled Caissons, Spread Footings, or Structures

- Refer to PP3.6.4.8 for base plate design.
- Refer to PP3.6.4.9 for anchor bolt design.
- Minimum fillet weld size = 3/8 inch

(3) Steel Post Embedded in Drilled Caisson or Spread Footing with Pedestal

- Fatigue design for welded studs, for ground mounted walls, shall be evaluated at two million cycles, non-redundant, Category C. Refer to AASHTO Standard Specifications for Highway Bridges, 17th Edition, Section 10.38.5.1.1 and 10.38.5.1.2.
- Post embedded in Drilled Caisson
 - The centerline of the top welded stud must be located at a depth where the caisson reinforcement is fully developed.
 - The post and welded studs must extend beyond the centerline of the top stud so the applied forces can be transferred into the drilled caisson.
- Post embedded in Spread Footing
 - The centerline of the top welded stud must be located a minimum of 6 inches below the top of spread footing.
 - The post and welded studs must extend beyond the centerline of the top stud so the applied forces can be transferred into the spread footing.

(4) Steel Pipe Post Embedded in Drilled Caisson or Spread Footing (with or without pedestal)

- Provide steel pipe posts when the intersecting angle between adjacent panels is greater than 78 degrees and less than 160 degrees.
- Provide structural steel tubing conforming to ASTM A53, Grade B, Type E. [$F_y = 35$ ksi]

- Design Conditions:
 - Design Condition 1:
Design post for a wind direction normal to the post with an effective width equal to the post spacing multiplied by the sine of the one-half the angle between the centerline of the panels. Design the pipe post for flexure, shear, and deflection in accordance with the requirements of AASHTO and Design Manual, Part 4. In addition, the attached flange plates and weld must be checked for combined bending and shear stresses.
 - Design Condition 2:
Design post for a wind direction normal to the panel with an effective width equal to one-half the post spacing applied to only one side of the post. Design the pipe post for flexure, shear, combined torsion and shear, and deflection in accordance with the requirements of AASHTO and Design Manual, Part 4. In addition, the attached flange plates and weld must be checked for combined bending and shear stresses.
- For additional information refer to PP3.6.4.7, item (c)(3).

3.6.4.8 Base Plates

- (a) Base plates shall be designed for bending due to applied compression and tension anchor bolt forces. The anchor bolt force may be assumed to be distributed out at an angle of 45 degrees from the center of the anchor bolt.
- (b) Determine base plate thickness in accordance with AASHTO Standard Specifications for Structural Supports of Highway Signs, Luminaries and Traffic Signals, 4th Edition, 2001 including Interims through 2006, Section 5.8.
- (c) Minimum base plate thickness shall be 3/4 inch.
- (d) Support all base plates on washers and leveling nuts. Place non-shrink grout between the bottom of the base plate and supporting component. Pack grout into place, do not pour or inject. Due to construction issues related to the acceptable placement of the grout, the non-shrink grout shall not be considered as a load-carrying element.
- (e) Base plates for ground mounted sound barrier walls designed and detailed in the Sound Barrier Standard Drawings are not designed to support the vertical load of the precast concrete panels.
- (f) Base plates for structure mounted sound barrier walls designed and detailed in the Sound Barrier Standard Drawings are not designed to support the vertical load of the precast concrete panels even though the panels rests on the base plate. The reason for this is that the panel is continuously supported on top of the barrier or retaining wall, thus the base plate is not induced to any additional loading.
- (g) Alternate base plate designs are permitted which include the vertical load of the panel supported on the base plate. Alternate designs must be accepted by the Chief Bridge Engineer.

3.6.4.9 Anchor Bolts

- (a) Anchor Bolts shall be designed in accordance with AASHTO Standard Specifications for Structural Supports of Highway Signs, Luminaries and Traffic Signals, 4th Edition, 2001 including Interims through 2006, Sections 5.12 and 5.17.
 - Allowable Compression Stress = $0.5F_y$ (Maximum)
- (b) Minimum anchor bolt diameter shall be 3/4 inch.
- (c) Anchor bolts shall be embedded to a depth sufficient to develop the yield strength of the anchor bolt. The minimum anchor bolt embedment length must extend to a depth where the supporting component reinforcement is fully developed, above and below the embedment length, and capable of transferring the applied forces.

- (d) Anchor bolts shall be cast-in-place and placed before placing concrete. Preformed holes and drilled holes are not permitted.
- (e) Adhesive anchors are not permitted.

3.6.4.10 Foundations

(a) Spread Footings

- (1) Spread footings shall be designed bearing on soil or rock in accordance with PP3.6.4.5.
- (2) Design of spread footings shall consider if the proposed ground line is level or sloping.
- (3) Design spread footings to include live load surcharge, if required.
- (4) Provide a minimum soil depth of 1'-6" above the top of footing.
- (5) Provide a minimum footing thickness of 1'-6".
- (6) Design footings for no uplift if supported on soil.
- (7) Spread footings designed and detailed in the Sound Barrier Standard Drawings use the following parameters:
 - Spread footings are designed bearing on soil with an allowable bearing pressure equal to 1.50 tons/square foot and a coefficient of sliding friction equal to 0.30.
 - Spread footings are designed for wind pressures equal to 20 psf and 28 psf.
 - Spread footings are designed for level ground.
 - Spread footings are designed for no uplift.
 - Spread footings are designed for no live load surcharge.
- (8) Alternate spread footing designs are permitted and are required if the foundation design parameters and site conditions differ from those indicated in the Standard Drawings. Foundation parameters must be accepted by the Department.

(b) Drilled Caissons (Shafts)

- (1) The design of laterally loaded drilled caissons shall account for effects of soil layering, variable groundwater level, loss of lateral support, cyclic loading, combined axial and lateral loading and sloping ground.
- (2) Drilled Caisson lengths shall be determined using COM624P or LPILE 5.0 computer program based on the site specific soil properties. An additional 3'-0" length must be added to the minimum caisson length determined by COM624P or LPILE 5.0 to account for freezing and thawing, weathering, and other shallow ground disturbances.
- (3) Maximum lateral design displacement at top of caisson = 1/2 inch
- (4) Maximum allowable vertical displacement = 1 inch
- (5) Design of caissons shall consider if the proposed ground line is level or sloping.
- (6) Design caissons to include live load surcharge, if required.
- (7) Provide a minimum factor of safety against overturning of 2.0.
- (8) Provide a minimum drilled caisson diameter of 2'-6".

- (9) Provide a minimum caisson length in soil equal to 3 times the caisson diameter.
- (10) Drilled caissons designed and detailed in the Sound Barrier Standard Drawings use the following parameters:
 - Drilled caissons are designed using the soil properties for the four soil types indicated.
 - Drilled caissons are designed for a wind pressures equal to 28 psf.
 - Drilled caissons are designed for level ground.
 - Drilled caissons are designed for no live load surcharge.
- (11) Alternate drilled caisson designs are permitted and are required if the foundation design parameters and site conditions differ from those indicated in the Standard Drawings. Foundation parameters must be accepted by the Department.
 - The designer should take full advantage of the site specific conditions rather than rely on the limited soil types indicated in the Standard Drawings in order to reduce the required caisson lengths and overall construction cost.

3.6.4.11 Offset Walls

- (a) Support offset wall panels on a cast-in-place spread footing.
- (b) The offset wall stability analysis shall be based on a four-panel unit length. The stability analysis shall not include the spread footing since the wall is not rigidly attached to the spread footing. The stability analysis shall be made at the interface of the panel and footing. The minimum factor of safety against overturning and sliding shall be in accordance with PP3.6.4.5 for soil. Passive soil resistance is to be neglected.
- (c) The bearing stress, f_b , at the interface of the panel and footing shall not exceed $0.30 f'_c$. Negative bearing stress (i.e. uplift) is not permitted.
- (d) Precast concrete panels shall be designed in accordance with PP3.6.4.6. Panels shall be designed for individual backfill lift heights and/or maximum soil height differential between the two sides of the panel. Individual backfill heights shall be indicated and shown on the contract drawings.
- (e) Panel Connections
 - (1) Connect the panels together using 3/8 inch 7 x 19 stainless steel (Type 302 or 304) flexible wire rope (aircraft cable) with a minimum breaking strength of 12 kips and 1/2 inch stainless steel cap screws.
 - (2) Design the connection for the Group Loads indicated in PP3.6.4.5, item (a) in conjunction with a two degree wall tilt.
 - (3) Provide a minimum of two cable connections for each panel to panel connection. Provide a minimum of three cables connections for the end panel to adjacent panel connection.
 - (4) End panel shall be connected to the spread footing in accordance with the details shown on BC-780M.
- (f) Spread Footings
 - (1) Spread footings shall be designed bearing on soil or rock in accordance with PP3.6.4.5.
 - (2) Design of spread footings shall consider if the proposed ground line is level or sloping.
 - (3) Design spread footings to include live load surcharge, if required.
 - (4) Provide a minimum soil depth (panel embedment depth) of 2'-0" above the top of footing.

- (5) Provide a minimum footing thickness of 1'-0".
- (6) Provide a minimum footing width of 3'-0".
- (7) Design footings for no uplift if supported on soil.
- (8) Provide footing steps as required.
 - Minimum step height = 6 inches
 - Maximum step height = 2'-0"
- (9) Spread footings designed and detailed in the Sound Barrier Standard Drawings use the following parameters:
 - Spread footings are designed bearing on soil with an allowable bearing pressure equal to 1.50 tons/square foot and a coefficient of sliding friction equal to 0.30.
 - Spread footings are designed for wind pressures equal to 20 psf and 28 psf.
 - Spread footings are designed for level ground.
 - Spread footings are designed for no uplift.
 - Spread footings are designed for no live load surcharge.
- (10) Alternate spread footing designs are permitted if the foundation design parameters and site conditions differ from those indicated in the Standard Drawings. Foundation parameters must be accepted by the Department.

3.6.4.12 Plan Presentation and Design Items

Designs shall conform to Standards Drawings. The following information and details must be part of each submission:

- (a) Beginning and end wall stations.
- (b) Overall wall length.
- (c) Horizontal alignment of sound barrier.
- (d) Vertical alignment of sound barrier.
- (e) Stake-out sketch including work point coordinates.
- (f) Post spacing and type of post.
- (g) Elevations (as required) indicating the acoustic profile, existing ground line, proposed ground line, top of traffic barrier and top of retaining wall. Elevations shall be given at a minimum 50'-0" interval. Provide additional elevations as required.
- (h) Elevations of top and bottom of spread footings and drilled caissons, type of foundation, spread footing maximum allowable and design bearing pressures, drainage location; depth, and extent of any unsuitable material to be removed and replaced.
- (i) Right-of-way limits.
- (j) Construction Sequence.
- (k) Quantity table showing the estimated sound barrier area in square feet (show method of payment).

- (l) Boring logs, when applicable.
- (m) Approximate top of rock elevations, when applicable.
- (n) Approximate ground water elevations, when applicable.
- (o) Prepare design calculations for structural design, foundation design, post to foundation design or post to barrier anchor design (post to panel securing). Design calculations are not required if the designs and details are taken directly from the Standard Drawings.
- (p) Design and dimension tables.
- (q) Details of fit between panels and posts.
- (r) Details of post connections to supporting components (i.e. spread footing, drilled caisson, traffic barrier, and retaining wall).
- (s) Emergency access, maintenance access.
- (t) If no approved wall type exists, all key structural foundation and acoustic items shall either be detailed on the contract plans or clearly specified in the construction specifications.
- (u) Provide a signed statement from the District Environmental Manager indicating that the acoustic requirements of the proposed sound barrier have been reviewed and accepted.
- (v) Limits of penetrating concrete stain.
- (w) Any other information required to construct the sound barrier wall.

3.6.4.13 Special Provisions

The following information must be specified in the contract special provisions:

- (a) Type of permitted alternate wall types.
- (b) Type of permitted post types. (Precast concrete or steel)
- (c) Type of Architectural Surface Treatments on the residential and roadway sides on the precast concrete sound barrier panels and posts.
- (d) Color of the Integral Pigmentation for the Precast Concrete sound barrier panels and posts.
- (e) Color of joint sealing material and/or caulking compound and non-shrink grout.
- (f) Color of Penetrating Concrete Stain.
- (g) Paint Color of Steel components.
- (h) Provide Federal Color Numbers in accordance with Federal Standard Number 595A or 595B.
- (i) and any other information that may be required to construct the sound barrier wall.

3.6.4.14 Acoustic Performance Specifications

- (a) Sound barrier panels shall achieve a minimum Sound Transmission Class (STC) of 25 as measured in accordance with ASTM E90-99.
 - (1) Precast concrete sound barriers panels with a minimum structural thickness equal to or greater than 4 inch will achieve a Sound Transmission Class (STC) of at least 25 and therefore do not require an Independent Laboratory Test Report.
 - (2) Precast concrete panels with a structural thickness less than 4 inch or panels which are constructed from other materials must be approved by the Department using the New Product Evaluation Process, prior to bidding, and must include an Independent Laboratory Test Report indicating the Sound Transmission Class (STC) achieves a value of 25 or more.
- (b) Sound absorptive panels shall achieve a minimum Noise Reduction Coefficient (NRC) of 0.70 as measured in accordance with ASTM C423-02a.
 - (1) Sound absorptive panels must be approved by the Department using the New Product Evaluation Process, prior to bidding, and must include an Independent Laboratory Test Report indicating the Noise Reduction Coefficient (NRC) achieves a value of 0.70 or greater.

3.6.4.15 Submissions

Design and construction submissions shall be in accordance with PP1.9 and PP1.10 for ground and structure mounted sound barriers with the following modifications:

- (a) The TS&L Report and Structure Geotechnical Foundation Report for ground mounted, retaining wall mounted, and moment slab mounted sound barrier walls shall be submitted concurrently. The submission requirements shall be in accordance with PP1.9.3 for TS&L and PP1.9.4 for Foundations.
- (b) Each submission shall show the appropriate data (PP1.9.4 Foundations omitted) for Bridge Mounted sound barrier walls.
- (c) Responsibilities as delineated for Bridge Submission in PP1.9 and PP1.10 are also applied to ground mounted sound barrier walls.
- (d) Shop drawings are required in accordance with PP1.10.2.3.

3.6.4.16 Design Build

- (a) Design Build projects shall be prepared, if directed by the Department, for the Ground Mounted Post and Panel Sound Barrier Walls which may allow the Contractor to determine the appropriate post type, post spacing, and foundation type based on the information provided in the contract documents. For additional information refer to PP1.11.
- (b) Design Build projects shall be not be prepared for Ground Mounted Offset Sound Barrier Walls unless directed by the Department.
- (c) Design Build projects shall be not be prepared for Structure Mounted Sound Barrier Walls since the post spacing and wall elements shall be set by the Design Engineer unless directed by the Department.

3.6.5 Pedestrian Structures and Bridges on Shared Use Trails

Enhancement and rails to trails pedestrian structures, involving both new construction and rehabilitation, shall be reviewed for critical areas such as the applicable material specifications, deflections, design loads, member dimensions, fabrication details, connections, and special provisions for erection and construction. If a proprietary product does not meet the criteria, it must be submitted and evaluated through the Department's Product Evaluation Process. Pedestrian and shared use trail structures may be produced only by Bulletin 15 approved fabricators meeting the requirements listed herein. The Department will provide full time in-plant quality assurance inspection during fabrication.

All pedestrian and shared use trail structures may be categorized in one of three groups as follows:

- (a) Group I – Structures located on or over Department right-of-way
- (b) Group II – Structures not located on or over Department right-of-way but crossing a public roadway (roadway owned by another local or state agency)
- (c) Group III – Structures not on or crossing any public roadway (i.e.; structures in parks or crossing railroads)

3.6.5.1 Group I – Structures Located on or over Department Right-Of-Way

All pedestrian and shared use trail structures located on or over Department final right-of-way shall conform to the DM-4 policies, procedures, and specifications, including appropriate design submissions. Certification acceptance (Stewardship and Oversight Agreement) procedures shall be followed. In all cases the designer shall stamp and seal the structure plans as per PP1.6.3.1, and the Bridge Engineer shall review and approve the plans “For Structural Adequacy Only.” Publication 408 specifications shall be used for construction and materials.

In every case, the structure must be competitively bid and allow multiple manufacturer’s bridge types. These structures may be bid as designed with alternates or they may be bid as a Design-Build project. Highlights and exceptions to the specifications for these bridges are as follows:

- (a) Department criteria must be followed. Note the following requirements for:
 - (1) Redundancy (D1.3.4) – A redundancy analysis will be required for non-redundant structures
 - (2) Deflection (D2.5.2.6.2) – $L/1000$ for spans; $L/375$ for cantilever arms
 - (3) Live Load (D3.6.1.6) – Pedestrian load of 90 psf to be applied
 - (4) Inspection requirements (Pub 238) – Comprehensive inspection at 2-year maximum intervals
 - (5) Fatigue detail categories restrictions (D6.6.1.2.6P) – Category C or better detail must be provided
 - (6) Bearings and Joints (D14) – Method A used for laminated neoprene bearings
 - (7) Fracture Critical Members (PP1.7.7 (13))
 - (8) Construction and Fabrication (Publication 408) – Bridge fabricator must have current AISC certification to the Major Steel Bridge category with Fracture Critical endorsement.
- (b) ASTM A500 (indicate Grade) and A847 materials may be used.
- (c) ANSI/AWS/D1.1 is applicable for welding structural shapes to tubular members. (also see 3.6.5.1, item (e))
- (d) 100% of welds on main load carrying tubular members shall be non destructively tested as follows:
 - (1) Full penetration groove welds in butt joints shall be radiographically tested.
 - (2) Full penetration groove welds in T and corner joints shall be ultrasonically tested (UT). For material less than 5/16 in. thick, UT procedures shall be submitted to the Chief Structural Materials Engineer for approval prior to use.
 - (3) Partial penetration groove welds and fillet welds shall be magnetic particle tested.
- (e) Main load carrying member components of A709 steel subject to tensile stress shall meet the supplementary notch toughness requirements for the longitudinal Charpy V-notch test specified for Zone 2 in Table S1.2 (non-fracture critical) or S1.3 (fracture critical) of the applicable ASTM material specifications. A500 and A847 tubular members shall meet the requirements stipulated in the Tables for A709, Grade 50 material. Tubular members shall be tested at “P” piece frequency (sampled at one end of each length of tubing supplied) for fracture critical members, and at “H” (heat lot) frequency for non-fracture critical members, all in accordance with ASTM A673/A673M.

- (f) SMAW, SAW, FCAW, and GMAW are approved welding processes, except that FCAW-S (self-shielding) and GMAW-S (short circuit arc transfer) will not be accepted for any welding.
- (g) All Weld Procedure Specifications (WPS's) shall be submitted to, and approved by, the Chief Structural Materials Engineer prior to production welding, including tack welding. Prequalification of weld procedure specifications for welds on tubular members will be determined in strict compliance with Chapter 3 and Annex H of the latest edition of ANSI/AWS/D1.1. For welded non-tubular structures, welding and weld procedure qualification test should conform to ANSI/AWS/D1.5.

3.6.5.2 Group II – Structures not Located on or over Department Right-Of-Way but Crossing a Public Roadway

Review, approval and bidding requirements are the same as Group I. Highlights and exceptions to design criteria for these structures not on or over Department right-of-way but crossing a public roadway are as follows:

- (a) A comprehensive structure inspection is completed as per Publication 238M every two years.
- (b) ASTM A500 (indicate Grade) and A847 materials may be used.
- (c) ANSI/AWS/D1.1 is applicable for welding structural shapes to tubular members. (also see 3.6.5.2, item (e))
- (d) 100% of welds on main load carrying tubular members shall be non-destructively tested as follows:
 - (1) Full penetration groove welds in butt joints shall be radiographically tested.
 - (2) Full penetration groove welds in T and corner joints shall be ultrasonically tested (UT). For material less than 5/16 in. thick, UT procedures shall be submitted to the Chief Structural Materials Engineer for approval prior to use.
 - (3) Partial penetration groove welds and fillet welds shall be magnetic particle tested.
- (e) Main load carrying member components of A709 steel subject to tensile stress shall meet the supplementary notch toughness requirements for the longitudinal Charpy V-notch test specified for Zone 2 in Table S1.2 (non-fracture critical) or S1.3 (fracture critical) of the applicable ASTM material specifications. A500 and A847 tubular members shall meet the requirements stipulated in the Tables for A709, Grade 50 material. Tubular members shall be tested at "P" piece frequency (sampled at one end of each length of tubing supplied) for fracture critical members, and at "H" (heat lot) frequency for non-fracture critical members, all in accordance with ASTM A673/A673M.
- (f) SMAW, SAW, FCAW, and GMAW are approved welding processes, except that FCAW-S (self-shielding) and GMAW-S (short circuit arc transfer) will not be accepted for any welding.
- (g) All Weld Procedure Specifications (WPS's) shall be submitted to, and approved by, the Chief Structural Materials Engineer prior to production welding, including tack welding. Prequalification of weld procedure specifications for welds on tubular members will be determined in strict compliance with Chapter 3 and Annex H of the latest edition of ANSI/AWS D1.1. For welded non-tubular structures, welding and weld procedure qualification test should conform to ANSI/AWS/D1.5.
- (h) The redundancy requirement (D1.3.4) may be waived
- (i) PennDOT specifications for neoprene bearings and expansion joints shall be used.
- (j) Bridge fabricator must have current AISC certification to either the Simple Steel Bridge Structure category or the Major Steel Bridges Category.
- (k) All fatigue details must be designed in accordance with AASHTO. (The Category C requirement may be waived.)
- (l) Deflection must meet the criteria contained in Section 5 of the AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges. DM-4 deflection criteria may be waived.

3.6.5.3 Group III – Structures not on or Crossing any Public Roadway (I.E.: Structures in Parks or Crossing Railroads)

For locally sponsored and owned pedestrian structures involving state and/or federal funding which are located off of the Department final right-of-way and which do not cross a public road, the local owner may accept review responsibility. Examples of this would be a pedestrian trail bridge in a state or local park over a small creek or a pedestrian bridge over a railroad. In these cases, the AASHTO minimum criteria for design (see AASHTO Guide Specifications for Design of Pedestrian Bridges) may be used provided an independent check of the plans and computations for conformance to design criteria and structural adequacy is completed by a licensed Professional Engineer provided by the local owner. The designer shall stamp and seal the structure plans, and the review engineer shall sign and seal the plans using the following format:

Design reviewed by: Review Consultant's Name, Signature and Date The design review is for general conformance with AASHTO design and construction criteria and is not intended to relieve the designer of full responsibility for the accuracy and completeness of the plans.	PE Seal

If the local owner does not accept review responsibility, the Department may be asked to provide a review. In this case, Group II criteria for pedestrian structures must be followed, and the District Bridge Engineer will approve the plans "For Structural Adequacy Only." In these cases, inspection will be required from the local owner on a two-year cycle.

3.6.5.4 Fabrication and Construction Specifications

Designers and reviewers should be aware that the PennDOT Publication 408 is very specific in its specifications for construction, and may require prequalification for fabricators and/or specific fabrication practices. If a local project is designed using AASHTO only, the designer must provide special provisions to allow construction practices which deviate from Publication 408. In addition, if the structure crosses over a private entity such as a Railroad, all supplemental design requirements of that entity must be met.

3.6.5.5 Bridges with Fracture Critical Members

For those pedestrian bridges with fracture critical members (FCM), FCM provisions (See PP1.7.7, Note 13) will continue to be required for structures over public roadways and significant water crossings (waterway not able to be traversed by foot during normal flow). Please note that special provisions will need to be developed by the designer to allow any construction and material exceptions selected by the local municipality.

3.6.5.6 Usage of Recycled Bridges

Recycled (used) bridges may be acceptable provided that the structure meets the following conditions:

- A complete inspection has been performed
- The material certifications are acceptable, or physical testing has been completed
- New connection material is utilized if the bridge is reconstructed or reassembled (new bolts, etc.), and
- The bridge is accepted by the District Bridge Engineer.

3.6.6 Usage of Unapproved Products

Any item not covered by the established standards, criteria or specifications and seeking approval in Bulletin 15 must go through the Bureau of Project Delivery.

With the increasing volume of new products being marketed for incorporation into Department construction projects, it is important that established procedures for evaluation of new products be followed. See Publication 51, Chapter 3C.

Use Master Policy Statement No. 418 for orderly product evaluation.

MPS No. 418 has been in effect for many years and was revised in August 1989 only to reflect Act 101 (1988), which requires mandatory recycling.

It is mandatory that new products should not be incorporated into Department projects unless approved for inclusion by the Chief Bridge Engineer.

3.6.7 Bridge Inspectability

For design requirements see D2.5.2.2.

To assist in reviewing deck girder/truss designs for inspectability using PennDOT underbridge crane, use the following guidelines and Figure 3.6.7-1.

1. Provide adequate lateral clearance from bridge superstructure to obstructions to permit crane boom deployment.
 - (a) Horizontal clear distance between dual bridges or an obstruction "A" > 10 ft. minimum (> 15 ft. desirable). Clearances from electric power lines are critical.
2. For maximum horizontal reach of the crane under the bridge:

Crane boom B-3 (telescoping) must be deployed horizontally. Therefore:

- (a) Vertical distance "D" < 20 ft. and/or
- (b) Barrier/fences/sound barrier height "B" < 9 ft. and/or
- (c) Depth to bottom of superstructure "C" < 11 ft.
- (d) Horizontal distance from center of truck to outside edge of structure "E" < 16 ft.

With the above instructions:

Maximum Horizontal Reach "H" = 44 ft.

3. Many bridge configurations outside the above restrictions are inspectable with a crane, but reach is compromised. For example, a deep girder with:
 - (a) Barrier Height < 42 in.
 - (b) Depth to bottom of superstructure "C" < 22 ft.
 - (c) Truck adjacent to barrier "E" = 7 ft.

The maximum horizontal reach "H" is limited to 35 ft.

4. The bridge should be considered inspectable if reasonable access is provided within each bay of the bridge superstructure. Currently, Federal regulations require hands-on (within arms length) inspections every two years for fracture critical members. For non-fracture critical members, this requirement is extended to every six years, i.e. every third inspection. Therefore, every effort should be made to provide hands-on access to all fracture critical members via crane inspection. Provide other means of access (such as tie-off attachments for rope access) for inspection and maintenance in areas without hands-on access for fracture critical members whenever prudent and cost effective.

5. Contractor alternates and Value Engineering Proposals must provide the same level of inspectability as original designs.
6. These guidelines are to be applied assuming the use of a crane with a 50 foot horizontal reach to ensure that the bridge's inspectability is not limited to a single crane and that generic design specifications are prepared.
7. For through-truss bridges or those with other obstructions (power lines, buildings, etc.), restricted boom deployment may limit crane use. For some situations, inspectability by crane may have to be determined in the field.
8. If the District would like the Bridge Design and Technology Division to review a specific bridge configuration, especially at the TS&L stage, submit a scaled drawing $1/4" = 1'-0"$ of the bridge with obstructions.

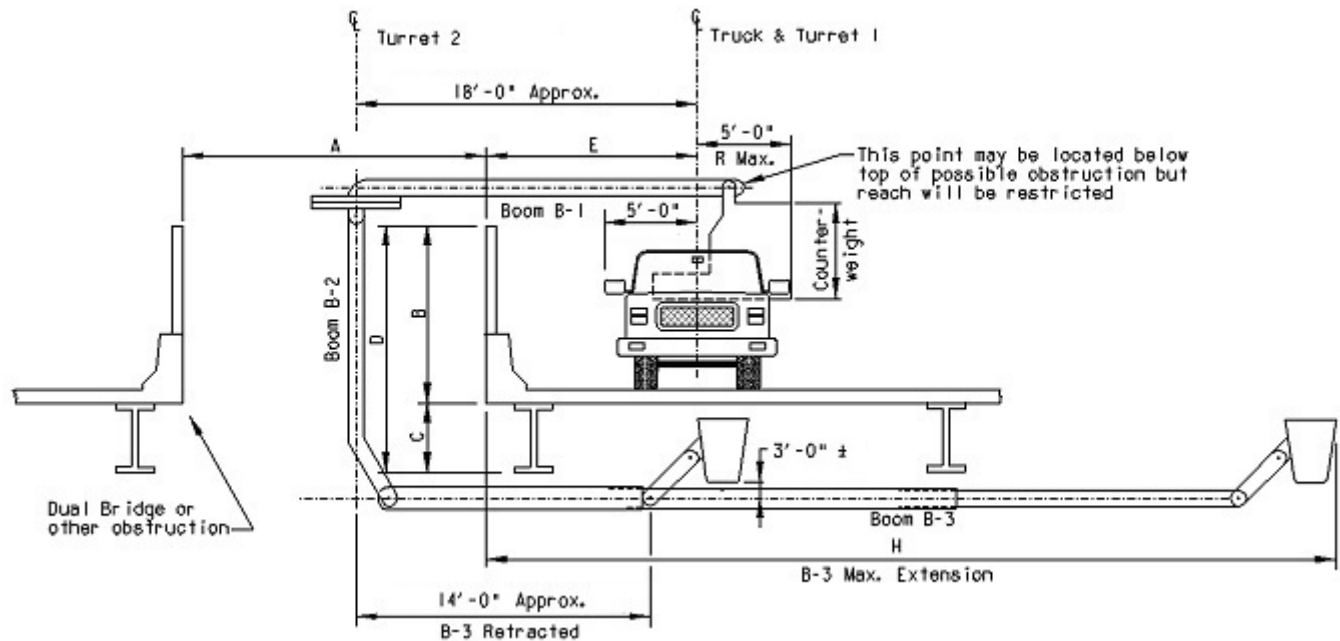


Figure 3.6.7-1 – PennDOT Underbridge Inspection Crane

3.6.8 Adhesive Anchors

Do not use adhesive anchors in a tension application for permanent installations.

3.6.9 Self Consolidating Concrete

Consideration can be given to the use of self consolidating concrete in piers, caissons, rehabilitation projects to aid in constructability and in precast members to aid in fabrication.

3.7 SPECIAL PROVISIONS

In the event that a design requirement or direction applying to any item of a project is not contained in, or deviates from, Publication 408, its supplements, or this Manual, the designer of the project shall write a special provision for its inclusion in the proposal. Such special provisions shall be in the imperative mood.

When alternates by contractors are permitted and the designer deviates from established design criteria (LRFD, Standard Drawings or this Manual), a special provision specifying the design criteria used shall be included in the contract documents. Any deviation from the established design criteria shall be approved by the Chief Bridge Engineer.

Refer to Publication 51 for guidelines concerning the preparation of contract proposals.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1

PART A: POLICIES AND PROCEDURES

CHAPTER 4 – BRIDGE ECONOMICS

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4.1 BRIDGE ECONOMICS - GENERAL

4.1.1 Cost Effective Bridges

There are many factors which will influence the cost of a bridge, such as type of superstructure, type of substructure, construction material, fabrication, transportation, erection, maintenance, traffic protection, etc. The cost of these factors changes with time, along with the cost relationship among them. The combined cost for superstructure and substructure for a given bridge site will determine the most economical bridge.

4.1.2 Economic Consideration

The following factors should be considered in determining the bridge cost.

1. Geometry:

Work with the roadway designer in providing the simplest possible geometry on the bridge. A 90° skew bridge will not only simplify the design, but will offer the most economical construction. Similarly, variable width ramps, superelevation or superelevation transition, and simple, compound or spiral curves on the bridge should be avoided where practicable. Consideration of future widening for increased traffic demands may be considered if desired.

Keep bridge geometry, including beam framing, as simple as possible.

Whenever possible, consideration should be given to replacing part of the bridge length with an embankment. Sometimes, combination of several short-span bridges and embankments may be more economical than one long-span bridge.

2. Hydraulic Requirement:

Hydraulic and regulatory requirements will dictate the minimum span length and minimum bottom of the beam elevation for bridges crossing streams or rivers.

3. Type of Structure:

A precast or prefabricated bridge is usually more economical than a cast-in-place bridge.

In selecting a bridge type, the designer should use the systematic elimination process. Start this process with all possible options as stated in PP4.1.3(a)1, then proceed to next level in PP4.1.3(a). Continue this process until the most economical and practicable type of structure for the required bridge length is found. This should not be construed as a requirement that all possible types of culverts or bridges should be cost analyzed in selecting a bridge type for a bridge location. Some of the options can be eliminated due to impracticality, proven uneconomical bridge type or known problems related to the bridge type(s) under consideration as per PP1.9.3.3.2. A preliminary cost estimate may be performed to screen the first two or three most economical bridge types that survive the process of logical elimination.

Where an automated design and drafting system, such as BRADD, is used, the designer should strive to provide additional contract drawings for different bridge types for the site. However, the District Bridge Engineer may determine the extent to which these additional contract drawings are needed based on past contractor alternate bid records, new technology, geometry, right-of-way, under clearance, utilities, etc.

Refer to PP4.1.3 for superstructure selection.

Refer to PP4.1.4 for substructure selection.

4. Maintenance Cost:

Maintenance costs such as painting, repairs and other anticipated costs should be anticipated and may be considered in addition to the initial bridge construction cost.

5. Transportation, erection and fabrication cost:

Consider costs which are particularly critical for special bridge types. For example, superbeams, with inherent transportation problems, will require a police escort.

6. Social Need:

Local community needs should be considered. These needs should be identified at the environmental clearance stage. All possible bridge types which can be constructed at the site under consideration should be offered to the public. At public meetings, a commitment should not be made for one specific bridge type.

7. Delivery and Availability:

Consider delivery time of alternate components and evaluate resulting cost impacts on community.

4.1.3 Selection of Superstructure Types

(a) The following structure types should be considered in developing a bridge superstructure.

1. Span less than 20 ft.:

In this span range generally, precast R.C. culverts or pipes, metal culverts or pipes, precast R.C. boxes, or precast R.C. slab bridges are considered more economical structures than cast-in-place R.C. box culverts. Cast-in-place arch culverts are rarely economical, but precast arches may be considered. Alternatives may be limited by class of highway, ADT, or ADTT. Where bedrock is within 3 ft. to 5 ft. of the streambed or that of the lower roadway for the grade separation structures, cast-in-place rigid frame structures may be economical.

2. Spans 20 ft. to 30 ft.:

In this span range, P/S plank beam, P/S box beam, or P/S channel beam bridges are generally more economical than steel I-beam bridge. Consideration should also be given to multiple precast R.C. boxes or culverts or multiple span cast-in-place box culverts in lieu of a single span bridge, but physical constraints and characteristics of the project site, such as debris potential and aquatic habitat, may need to be considered.

3. Spans 30 ft. to 90 ft.:

In this span range, P/S I-beam, bulb-tee beam, NEXT beam or box beam bridges with composite deck slab are generally more economical superstructures. In the recent history, steel structures in this span range have not been proven economical. However, changing market conditions and bridge site conditions (low under clearance, sharp skew with large width variation, etc.) could make steel bridges in this span range economically attractive.

4. Spans 90 ft. to 165 ft.:

In this span range, steel multi-girder, P/S I-beam or bulb-tee beam and P/S box beam bridges may be equally cost effective. The final selection should be based on the cost analysis for each bridge type for each location.

5. Spans over 165 ft.:

Bridges with span length over 165 ft. are more complex structures. Process of selecting the most economical type of structure will require that the designer develop preliminary design using different superstructure types, span arrangement and substructure types. Generally, for spans up to 250 ft., multi-girder steel bridge may be an economical type of bridge. Refer to PP4.3.1 for economy of steel structures. However, consideration should also be given to spliced pre-post-tension concrete member bridge, drop-in span, or segmental bridges, etc. Note that the Chief Bridge Engineer's approval is required at TS&L stage for the bridge types not covered in the Department standards or for which Department design specifications are not explicit. Bridges with fracture-critical non-redundant members should not be used.

- (b) The following guidelines may be of further help to the designer in selecting the most economical bridge:
1. Previous designs for the similar bridge environment may be used as a guide by the designers.
 2. Use of approved sophisticated design methods or softwares may aid in reducing cost of the structure. Refer to PP1.3.3 for selection of Design Methodology.
 3. Minimize the number of girders at the bridge cross-section. Refer to PP3.2.1 for minimum number of girders required.
 4. Designers should take advantage of higher material strength whenever possible.
 5. Optimize the number of spans.
 6. Use the maximum beam depth allowed by the underclearance.
 7. Optimize the dead load of the bridge.

4.1.4 Selection of Substructure Types

1. Abutments can be reinforced concrete cantilever type, tie-back wall type, or approved proprietary wall type. Prefabricated walls and abutments are generally more economical than cast-in-place concrete walls and abutments. Refer to PP3.3.3 for economic consideration for substructure type selection. In most cases, the cost of a prefabricated wall abutment is generally about 50% or more of the conventional reinforced concrete abutment. Note that long-term settlement and service life must be considered in any substructure type selected.
2. When a reinforced concrete cantilever substructure is used, shallow spread footing on rock or good founding material is usually the most economical foundation. However, potential settlement and potential scour depth may require a deeper foundation.
3. When suitable rock is available at an average depth of less than 10 ft. below the proposed bottom of footing, pedestal foundation or foundation which is made possible by removal of the overburden and backfilling with lean concrete or suitable material may be more economical than piling or drilled shafts. For depths greater than 10 ft., the piling is usually more economical than the drilled shafts. However, in special situations (where piles cannot be driven due to site conditions), augured micropiles or drilled concrete shafts have been proven to be more economical for 10 ft. to 30 ft. lengths. Where practical, an option of different foundation types shall be given to the contractor.
4. Integral abutments supported by piles may be economical and advantageous from a future maintenance point of view, since they eliminate the joints at the abutments. Refer to Appendix G.
5. Minimizing the number of substructure units is usually more economical where a deep foundation is contemplated.
6. Substructure units should be optimized in shape and size to ease construction and economize quantity. Special forms should be avoided unless it is for aesthetic or other special reasons. However, site conditions must be satisfied.

4.1.5 Cost Analysis

A life-cycle cost analysis may be employed in determining the cost of the bridge. Approximate cost analysis may be used to screen the types of bridges on the preliminary list.

The following list includes some (but not all) of the most common factors that should be considered in the life-cycle cost analysis.

1. First cost of the structure.
2. Salvage value of the structure. Note that in most cases salvage value is negative (i.e., removal cost).
3. Design and engineering cost.

4. Maintenance costs.
5. Traffic maintenance and protection costs, during maintenance, rehabilitation or replacement.
6. Painting cost.
7. Deck replacement cost.

4.1.6 Final List of Bridge Type

At least three bridge types which survive the logical selection process should be submitted in the TS&L submission, together with life-cycle cost analysis and a final recommended bridge type. A submission containing less than three bridge types is permitted per PP1.9.3.3.2.

For a major bridge, a minimum of two bridge types should be studied for each steel and concrete alternate designs. One bridge type per alternate may be accepted if a reasonable explanation is provided.

The Department policy is to encourage all possible contractor alternate designs in order to reduce bridge construction costs (PP1.10.1.1). Therefore, designers should submit (in the TS&L submission) the design and construction restriction requirement of these alternate designs for the later use in the development of special provisions for alternate designs. Designers may refer to Appendix D for important items which have to be addressed in alternate design special provisions.

4.1.7 Value Engineering

For value engineering concepts and theory, refer to Design Manual, Part 1C, Chapter 3, Section 3.3.A.6. Value engineering is applied after TS&L, foundation or final plans are developed using common and best engineering concepts and materials known at the time of the plan development. The new substituted item must provide equal quality, service and longevity with the same or less maintenance need as for the item it replaces.

Value Engineering/Acceleration Construction Technology Transfer (VE/ACTT) is a workshop to develop cost effective and constructible projects through: deploying value engineering concepts prior to the start of final design or developing final contract documents; construction of a project on paper prior to final design to develop design around constructability; evaluating concepts and opportunities to expedite construction, and making preliminary design decisions to minimize permitting, design and construction time and costs. For projects where a Design VE is required and a VE/ACTT session has been held, the VE/ACTT satisfies the Design VE requirement per Design Manual, Part 1X, Appendix R, Section R.1.2.

4.2 LIFE-CYCLE COSTS

Currently the Department utilizes the “First Cost” of the structure (for all materials) as the Life Cycle cost.

For bridges on Very Low Volume Roads, it is permissible to perform a life cycle cost analysis of the decking to justify use of alternate deck types.

4.3 STEEL GIRDER BRIDGES

4.3.1 Guidelines for Economical Steel Girder Bridges

Many factors influence the cost of a steel girder bridge, including (but not limited to) type of material, type of substructure, amount of material, fabrication time (the number of detail pieces and shop operations involved), transportation and erection. The cost of these factors changes with time, along with the cost relationship between them. Therefore, the rules used to determine the most economical type of steel girder on one bridge must be reviewed and updated for the next bridge.

The guidelines listed below are taken from the summary of "Economical Steel Plate Girder Bridges", Engineering Journal, AISC, 2nd Quarter, 1984, pp. 89-93. The designer must evaluate each of these guidelines for validity as it pertains to the specific structure in question. The designer should be aware that these guidelines are subject to change for numerous reasons. Also see NSBA document 12.1-2003 (Guidelines for Design for Constructability) for additional guidance.

These guidelines apply to spans up to 200 ft. in length, representing a majority of the bridge population, although some also apply to longer spans.

1. Unpainted ASTM A 709/A 709M, Grade 50W, weathering steel is the most economical design. Properly designed in the appropriate environment, weathering steel bridges are more economical than those requiring painting of the whole structure. Note that the use of unpainted weathering steel requires special evaluation and the Chief Bridge Engineer's approval.
2. Designs should use the fewest number of girders compatible with deck design and other factors. A girder spacing of 10 ft. is suggested as the minimum for economical results. *Note that a minimum of four girders per bridge is required.*
3. Web design can have a significant impact on the overall cost of a plate girder. From the standpoint of material costs, it is usually desirable to make girder webs as thin as design considerations will permit. However, this may not always produce the greatest economy since fabricating and installing stiffeners is one of the most labor intensive of shop operations. The following guidelines are provided for the use of stiffeners:
 - (a) Transverse stiffeners (except diaphragm connections) should be placed on only one side of the web.
 - (b) Longitudinal stiffeners used in conjunction with transverse stiffeners on longer spans with deeper webs should preferably be placed on the opposite side of the web from the transverse stiffener. Where this is not possible, such as at intersections with cross-frame connection plates, the longitudinal stiffener should not be interrupted for the transverse stiffener.
4. Designs with web thickness which varies by field section are suggested.
5. Longitudinally stiffened designs should not be considered for spans less than 300 ft..
6. Use no more than three plates (two shop splices) in the top or bottom flange of field sections up to 130 ft. long. In some cases, a single flange plate size should be carried through the full-length of the field section.
7. Flange plates represent a significant portion of material costs. The amount of labor involved in fabricating flanges can vary significantly as a result of design. The most efficient way to make flanges is to butt weld together several wide plates of varying thickness received from the mill. After welding and non-destructive testing, the individual flanges are "stripped" from the full plate. This reduces the number of welds, individual run-off tabs to both start and stop welds, the amount of material waste and the number of x-rays for non-destructive testing. Therefore, it is preferable to keep flange widths constant within an individual shipping length by varying material thickness as required. This also makes it easier to use metal stay-in-place deck forms. This may not always be possible in girder spans over 350 ft. where a flange width transition may be required in the negative bending regions. Because plate is most economically purchased in widths of at least 48 in., it is best to repeat plate thickness as much as possible. An average of approximately 0.700 kips of flange material should be saved to justify the introduction of a flange splice.

When making flange transitions, consider two additional items:

- (a) It is good design practice to reduce the flange cross-sectional area by no more than approximately one-half of the area of the heavier flange plate to reduce the build-up of a stress at the transition.
 - (b) If a transition in width must be provided, shift the butt splice a minimum of 3 in. from the transition. This makes it much easier to fit run-off tabs, weld and test the splice and then grind off the run-off tabs.
8. Haunched girder designs should not be considered for most conventional cross-sections until spans exceed 400 ft..
 9. Omit bottom lateral bracing where such omission is permitted by LRFD. Evaluate the need for top flange lateral bracing during construction for span lengths over 200 ft. (see Standard Drawing BD-620M for details).
 10. Use elastomeric bearings or pot bearings in lieu of custom-fabricated steel bearings.

4.3.2 Rolled Beams

Unless otherwise directed, rolled beams shall be used where economical. Rolled beams should not be specified when the total camber exceeds 8 in. due to fabrication issues to achieve this camber. For this case, detail a plate girder with similar properties.

4.3.3 Composite Beams

Composite design is preferred for simple span and continuous bridges. Non-composite design may be used if it is more economical than composite design.

4.3.4 Intermediate Transverse Stiffeners

Straight girders may be designed with or without intermediate transverse stiffeners. For stiffened webs, the fabrication and handling costs must be considered in addition to material costs when determining the most economical design. Intermediate stiffeners may be designed on one or both sides of the web plates, but generally it is more economical to place stiffeners on one side of the web while placing on the opposite side only those stiffeners used also as connection plates.

4.3.5 Flange Plates

In the design of welded plate girder, changes in flange area can be accomplished by varying the thickness and/or the width of the flange plate by appropriate increments. A constant top flange width simplifies the forming of the deck. This should be weighed against economies accrued by varying the top flange width.

4.3.6 Girder Depth

When economically practical, the depth of fascia girders in multiple spans shall be as similar as possible. The girder depth shall be selected from an evaluation of girder depths considered in the preliminary design.

4.3.7 Field Section Length of Girders

The field section lengths of girders shall be determined by the designer through a thorough evaluation of site conditions, economy and hauling considerations. Before initiating final design using lengths greater than 70 ft., the designer must ensure that a hauling permit can be secured for the proposed length.

The designer is advised that superloads require special permits. A superload is defined as a vehicle with a gross weight over 201 kips (201,000 pounds), a length greater than 160 ft., or a width greater than 16 ft.. Therefore, a girder or group of girders with a weight of approximately 147 kips or greater, or a length of approximately 144 ft. or greater, would require vehicular transportation which is classified as a superload. Additional information concerning superloads can be found in PP1.13.2.

4.3.8 Structural Steels

There has been considerable discussion about the use of unpainted ASTM A 709/A 709M, Grade 50W or HPS-70W, steel in bridges. It is obviously important to be extremely cautious in areas of constant wetting, such as overpasses in urban areas. However, in areas where deicing salts are not a significant factor, or where structures are over streams or other areas where tunnel-effect entrapment of salt spray is not likely, ASTM A 709/A 709M, Grade 50W or HPS-70W, steel can probably still be of service. While the use of weathering steel can have a benefit with respect to long-term maintenance costs, it can also have a benefit in first-cost, which is the deciding factor in the current alternative bid situation. The first-cost savings come not only from the saving of the cost of the paint, but also savings in the surface treatment and handling of the finished girder in the fabrication process. For requirements that apply to the use of unpainted ASTM A 709/A 709M, Grade 50W or HPS-70W, steel, see D6.4.1.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 5 – REHABILITATION STRATEGIES

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5.0 DEFINITIONS AND NOTATIONS

The following definitions are used in this chapter:

Actual Stress Range – Fluctuation in stress experienced by the in-service member

Constant Amplitude Stress Ranges – Stress ranges of equal minimum and maximum stress and, therefore, stress range magnitude

Effective Stress Range – Constant amplitude stress range which is representative of a variable amplitude stress range spectrum

Fatigue Life – Number of cycles to failure at a specified stress range

GVW – Gross Vehicle Weight

PTF – Pennsylvania Traffic Factor

Probability of Failure – Statistical likelihood of a design criterion being exceeded

Refined Methods of Analysis – Analysis in which live load bending moments carried by each girder are computed by techniques as follows: three-dimensional finite element method, two-dimensional grillage analogy, or a simple line girder analysis using sophisticated live load distribution factors

Regression Analysis – Statistical analysis in which a curve is generated to approximate a random phenomenon

The following notations are used in this chapter:

A	=	Design fatigue life coefficient (PP5.1.1.1.3)
$(\Delta f)_D$	=	Calculated fatigue stress range (according to PP5.1.1.1.2a)
$(\Delta f)_e$	=	Effective stress range (PP5.1.1.1.2b)
$(\Delta f)_i$	=	Actual stress range of a random truck (PP5.1.1.1.2b)
$(\Delta f)_{ic}$	=	Calculated stress range of a random truck
N_{design}	=	Design fatigue life (PP5.1.1.1.3)
α	=	Ratio of the actual stress range caused by the passage of a particular vehicle to the calculated fatigue stress range caused by the passage of the same vehicle (PP5.1.1.1.2b) (This value is to be taken as 1.0. If a value different than 1.0 is to be used, it must be approved by the Chief Bridge Engineer.)
v_i	=	Frequency of occurrence i (PP5.1.1.1.2b) (decimal less than 1.0)
ϕ_i	=	$(\Delta f)_{ic}/(\Delta f)_D$ (PP5.1.1.1.2b) (Ratio of calculated stress range of a truck to the calculated design stress range)

5.1 FATIGUE DAMAGE OF STEEL BRIDGES

5.1.1 Load-Induced Fatigue

5.1.1.1 Load-Induced Fatigue Damage Assessment

Cumulative fatigue damage of uncracked members and fasteners subject to repeated variations or reversals of load-induced stress shall be assessed according to the provisions of PP5.1.1.1.1 through PP5.1.1.1.5. The list of detail categories and illustrative examples to consider in a fatigue damage assessment is shown in Table A6.6.1.2.3-1.

If cracks have already been visually detected, a more complex fracture mechanics approach for load-induced fatigue is required instead of the procedure outlined here. The fracture mechanics approach is beyond the scope of this presentation. Further, the expense and trouble of a fracture mechanics analysis may not be warranted. Generally, upon visual detection of cracking, the vast proportion (perhaps over 80%) of the fatigue life has been exhausted and retrofitting measures should be initiated.

5.1.1.1.1 Infinite Fatigue Life

If the factored live load stress range, $\gamma(PTF)(\Delta f)$, produced by the method described in PP5.1.1.1.2a is less than the constant amplitude fatigue threshold given in Table A6.6.1.2.5-3 using the Fatigue I limit state, the detail shall be considered to have infinite life.

5.1.1.1.2 Finite Fatigue Life

5.1.1.1.2a Calculated Fatigue Stress Range

- Approximate Method of Analysis – The factored live load stress range, $\gamma(PTF)(\Delta f)$, produced by the method given in the LRFD and the DM-4 (which is given in A3.4.1, D3.4.1, A3.6.1.4.1, A3.6.1.4.3b and D6.6.1.2.2) is considered the approximate method of analysis.
- Refined Methods of Analysis – Before a refined method of analysis can be used for a finite fatigue life evaluation, the approval of Chief Bridge Engineer must be obtained.

When using refined methods of analysis for finite fatigue life evaluation, the stress range shall be calculated by moving the loading condition, given in A3.6.1.4.1, across the bridge in the critical transverse position.

5.1.1.1.2b Effective Stress Range

The entire collection of stress ranges $((\Delta f)_i)$ caused by actual truck traffic that a structural detail will experience during its life is called the stress range spectrum. If all of the stress ranges are of equal magnitude (i.e., the amplitude from minimum stress to maximum stress is constant), the spectrum is called a constant-amplitude stress range spectrum. Since vehicles crossing the bridge are of various weights and axle configurations, the stress range spectrum experienced by a bridge detail is not of constant amplitude, but is a variable-amplitude stress range spectrum.

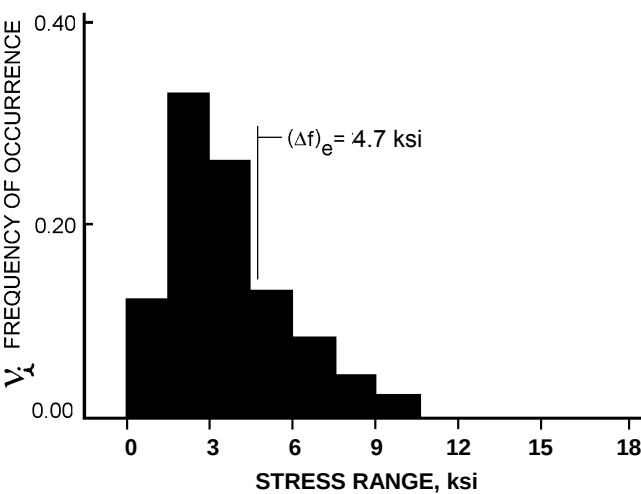
In terms of cumulative fatigue damage, an equivalent constant-amplitude spectrum will be used to represent the actual variable-amplitude spectrum experienced by a structural detail. Each type of spectrum consists of the same number of total stress range cycles (number of cycles = average daily truck traffic ($ADTT_{SL}$), as defined in A3.6.1.4.2 and D3.6.1.4.2, times the period of interest in days). The constant-amplitude stress range hypothesized to produce the same cumulative fatigue damage in the same total number of cycles as the variable-amplitude stress range spectrum is called that spectrum's effective stress range.

The effective stress range of a bridge detail shall be considered equal to

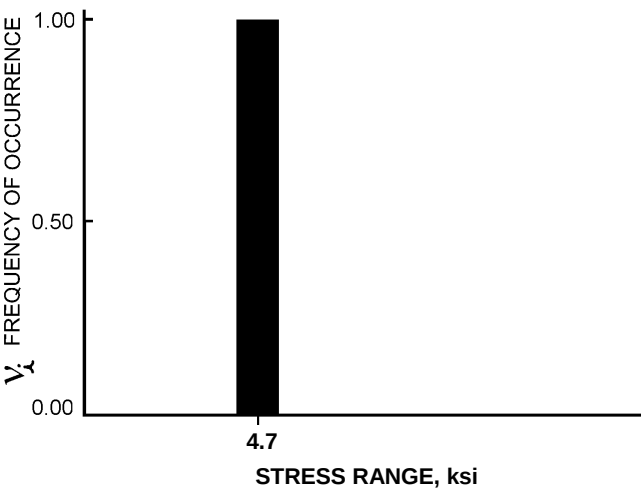
$$(\Delta f)_e = \left[\sum v_i (\Delta f)_i^3 \right]^{1/3} \quad (5.1.1.1.2b-1)$$

where v_i is the frequency of occurrence (a decimal less than 1.0) of stress range $(\Delta f)_i$, and $(\Delta f)_i$ is an experimentally measured (not calculated) value. This equation was developed from the fatigue damage accumulation model most commonly used for bridge engineering applications.

The concept of effective stress range is illustrated in Figure 5.1.1.1.2b-1. The histogram (or bar graph) shown in Part (a) of this figure represents the distribution of actual stress range that a particular bridge detail will experience during a period of time. Calculating an effective stress range according to Equation 5.1.1.1.2b-1 yields 4.7 ksi. The summation of the numbers of cycles represented in Part (a) of this figure is assumed to be equal to the $ADTT_{SL}$ times the period of interest in days.



(a) VARIABLE-AMPLITUDE HISTOGRAM



(b) CONSTANT-AMPLITUDE HISTOGRAM

Figure 5.1.1.1.2b-1 – Effective Stress Range Concept

In Part (b) of this figure, a histogram is shown of the same total number of cycles, but all with amplitude equal to 4.7 ksi. The accumulated fatigue damage which the distribution in Part (a) represents is assumed to be equal to that of the distribution in Part (b). In both histograms, the numbers of cycles represented is equal. In terms of accumulated fatigue damage, all of the variable amplitude stress range cycles of Part (a) can be replaced with an equal number of stress range cycles of constant amplitude equal to the effective stress range, as indicated in Part (b).

If the distribution of stress range (the v_i and $(\Delta f)_i$) is unknown, $(\Delta f)_e$ can be approximated for bridges in Pennsylvania as

$$\gamma(\text{PTF}) (\Delta f)_e = \gamma(\text{PTF}) \alpha (\Delta f)_D \quad (5.1.1.1.2b-2)$$

where:

$(\Delta f)_D$ = calculated fatigue stress range (according to PP5.1.1.1.2a)

α = ratio of actual stress range to calculated fatigue stress range; the value is to be taken as 1.0. If a value different than 1.0 is to be used, it must be approved by the Chief Bridge Engineer.

PTF = Pennsylvania Traffic Factor as given in D6.6.1.2.2

γ = load factor for fatigue given in A3.4.1 and D3.4.1.1

The approximation for effective stress range given in Equation 5.1.1.1.2b-2 is based upon the results of gross vehicle weight (GVW) surveys for Pennsylvania. The effective stress range can be more rigorously defined as follows:

$$\gamma(\Delta f)_e = \left[\sum v_i \phi_i^3 \right]^{1/3} \alpha (\Delta f)_D \quad (5.1.1.1.2b-3)$$

where:

v_i = frequency of occurrence of $(\Delta f)_i$

ϕ_i = $(\Delta f)_i / (\Delta f)_D$

α = ratio of actual stress range to calculated fatigue stress range; the value is to be taken as 1.0. If a value different than 1.0 is to be used, it must be approved by the Chief Bridge Engineer.

$(\Delta f)_D$ = calculated fatigue stress range according to PP5.1.1.1.2a

The 1970 FHWA Nationwide Loadometer Survey suggests that the summation of $v_i \phi_i^3$ be taken as 0.35, as a national average. This value is the basis for the past AASHTO (pre-LRFD) design provisions for fatigue. In recognition that GVW have been increasing since the 1970 FHWA survey, the LRFD design provisions for fatigue are based on an assumed summation of $v_i \phi_i^3$ equal to 0.422. However, the value can increase significantly for major truck arteries.

The most recent 1993 Pennsylvania GVW surveys of six locations indicated that for the worst location, I-80 in Clearfield County, the summation of $v_i \phi_i^3$ is 0.76. The Department has decided to use this worst case value of the summation of $v_i \phi_i^3$ equal to 0.76 (which will provide a PTF equal to 1.2) for freeways, expressways, major highways and streets with ADTT greater than 2500. A comparison of the 1970 FHWA, the 1973 Statewide PennDOT GVW surveys and 1993 PennDOT GVW survey of I-80 in Clearfield County is shown in Figure 5.1.1.1.2b-2. For major truck arteries, the value of 0.76 may be unconservative and detailed development using the principles outlined here is recommended. GVW distribution and corresponding cumulative damage tabulations of the 1973 Statewide Survey and 1993 I-80 in Clearfield County survey are given in Tables 5.1.1.1.2b-1, 5.1.1.1.2b-2 and 5.1.1.1.2b-3.

The factor α has been taken as 1.0. This value is inherent in the LRFD design provisions.

Table 5.1.1.1.2b-1 – Pennsylvania Statewide Cumulative Damage by Truck Type (1973 Data)

Gross Vehicle Weight Range	ϕ_i (GVW _i / GVW _o)	ϕ_i^3										
			$v_i(\%)$	$v_i \phi_i^3$	$v_i(\%)$	$v_i \phi_i^3$	$v_i(\%)$	$v_i \phi_i^3$	$v_i(\%)$	$v_i \phi_i^3$	$v_i(\%)$	$v_i \phi_i^3$
(kips)												
20.0-29.9	t	0.042	85.4	0.03573	48.11	0.02013	55.78	0.02334	24.52	0.01026	8.28	0.00346
30.0-39.9	0.486	0.115	14.14	0.01624	21.62	0.02482	38.45	0.04415	32.76	0.03762	15.48	0.01777
40.0-49.9	0.625	0.244	0.44	0.00107	14.06	0.03431	4.48	0.01093	24.82	0.06058	12.68	0.03095
50.0-59.9	0.764	0.446	0	0.00000	8.65	0.03855	1.28	0.00570	12.17	0.05423	15.16	0.06756
60.0-69.9	0.903	0.736	0	0.00000	4.86	0.03575	0	0.00000	4.53	0.03332	24.15	0.17765
70.0-79.9	1.042	1.130	0	0.00000	1.08	0.01220	0	0.00000	1.1	0.01243	19.43	0.21957
80.0-89.9	1.180	1.645	0	0.00000	1.62	0.02665	0	0.00000	0	0.00000	4.04	0.06646
90.0-99.9	1.319	2.297	0	0.00000	0	0.00000	0	0.00000	0.1	0.00230	0.71	0.01631
100.0-109.9	1.458	3.101	0	0.00000	0	0.00000	0	0.00000	0	0.00000	0.07	0.00217
110.0-over	1.528	3.566	0	0.00000	0	0.00000	0	0.00000	0	0.00000	0.02	0.00071
Number of Trucks per Type			226		185		156		995		4361	
Percentage of Truck Type			3.8		3.1		2.6		16.8		73.6	
$\Sigma v_i \phi_i^3$ by Truck Type* =			0.05304		0.19241		0.08412		0.21074		0.60261	

Total Cumulative Damage:

$$\begin{aligned}
 \Sigma v_i \phi_i^3 &= \Sigma [(\text{Percentage of Truck Type}) (\Sigma v_i \phi_i^3 \text{ by Truck Type})] \\
 &= 0.038(0.05304) + 0.031(0.19241) + 0.026(0.08412) + 0.168(0.21074) + 0.736(0.60261) \\
 &= 0.489
 \end{aligned}$$

*The quantity $\Sigma v_i \phi_i^3$ shown for each truck type represents the cumulative fatigue damage caused by the distribution of trucks in each truck type as a percentage of that caused by a like number of vehicles, all of the design gross vehicle weight.

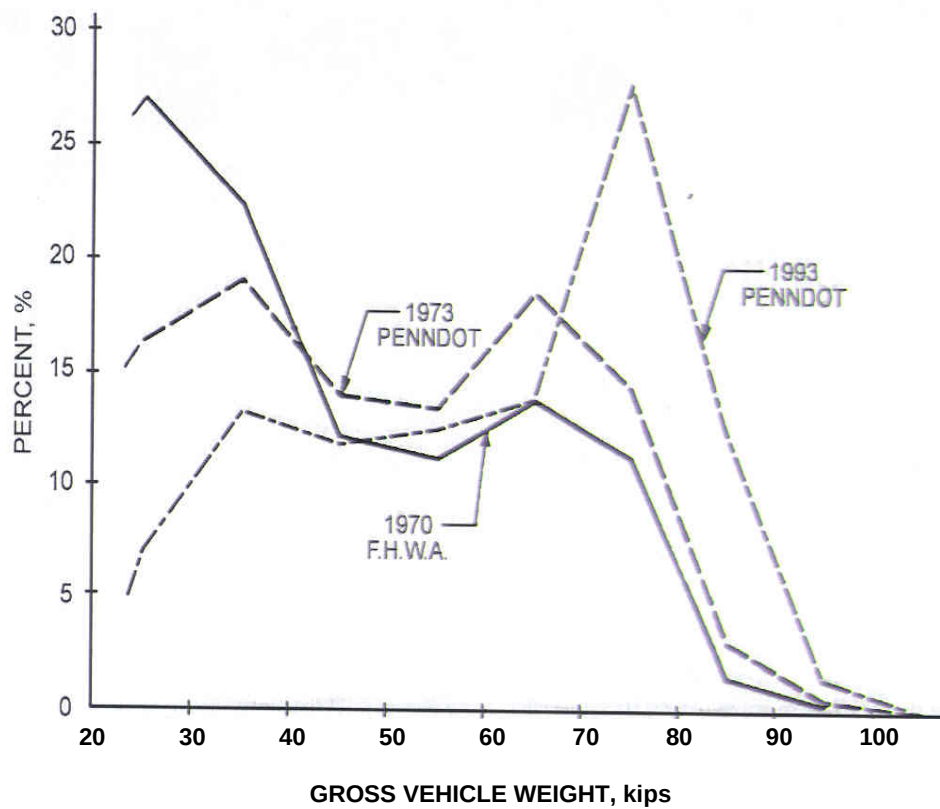


Figure 5.1.1.1.2b-2 – Loadometer Surveys

Table 5.1.1.1.2b-2 – GVW Distribution by Class of Vehicle (1993 Data)

Gross Vehicle Weight Range	Class of Vehicle											
	2		3		4		5		6		7 plus	
(kips)	Number	$v_i(\%)$	Number	$v_i(\%)$	Number	$v_i(\%)$	Number	$v_i(\%)$	Number	$v_i(\%)$	Number	$v_i(\%)$
20.0-29.9	9,370	74.98	240	12.27	2,213	49.47	520	3.05	6,666	31.97	34,455	4.88
30.0-39.9	2,352	18.82	1,182	60.43	1,472	32.91	497	2.92	6,176	29.62	89,760	12.70
40.0-49.9	544	4.35	500	25.56	622	13.91	442	2.59	4,544	21.79	83,393	11.80
50.0-59.9	190	1.52	30	1.53	136	3.04	1,010	5.93	2,084	9.99	92,286	13.06
60.0-69.9	21	0.17	3	0.15	22	0.49	4,020	23.58	777	3.73	101,121	14.31
70.0-79.9	18	0.14	0	0	4	0.09	9,575	56.17	529	2.54	201,183	28.47
80.0-89.9	0	0	1	0.05	2	0.05	936	5.49	68	0.33	93,431	13.22
90.0-99.9	1	0.01	0	0	1	0.02	42	0.25	6	0.03	9,911	1.40
100.0-109.9	0	0	0	0	1	0.02	3	0.02	1	0	671	0.09
110.0-119.9	0	0	0	0	0	0	0	0	0	0	284	0.04
120.0-129.9	0	0	0	0	0	0	0	0	0	0	107	0.02
130.0-139.9	0	0	0	0	0	0	0	0	0	0	54	0.01
140.0 plus	0	0	0	0	0	0	0	0	0	0	59	0.01
Sum	12,496	100.0	1,956	100.0	4,473	100.0	17,045	100.0	20,851	100.0	706,715	100.0
% in each class	1.64		0.26		0.58		2.23		2.73		92.56	

Table 5.1.1.1.2b-3 – Cumulative Damage by Truck Type (1993 Data)

Gross Vehicle Weight Range	ϕ_i (GVW _i /GVW ₀)	ϕ_i^3	$v_i \phi_i^3$ by Class of Vehicle					
(kips)			2	3	4	5	6	7 plus
20.0-29.9	0.347	0.042	0.0314	0.0051	0.0207	0.0013	0.0134	0.0020
30.0-39.9	0.486	0.115	0.0216	0.0694	0.0378	0.0034	0.0340	0.0146
40.0-49.9	0.625	0.244	0.0106	0.0624	0.0340	0.0063	0.0532	0.0288
50.0-59.9	0.764	0.446	0.0068	0.0068	0.0136	0.0264	0.0445	0.0582
60.0-69.9	0.903	0.736	0.0013	0.0011	0.0036	0.1735	0.0274	0.1053
70.0-79.9	1.042	1.130	0.0016	0	0.0010	0.6349	0.0287	0.3218
80.0-89.9	1.181	1.645	0	0.0008	0.0008	0.0903	0.0054	0.2175
90.0-99.9	1.319	2.297	0.0002	0	0.0005	0.0057	0.0007	0.0322
100.0-109.9	1.458	3.101	0	0	0.0006	0.0006	0	0.0028
110.0-119.9	1.597	4.075	0	0	0	0	0	0.0016
120.0-129.9	1.736	5.233	0	0	0	0	0	0.0010
130.0-139.9	1.875	6.592	0	0	0	0	0	0.0007
140.0 plus	2.014	8.168	0	0	0	0	0	0.0008
$\Sigma v_i \phi_i^3$ by Class of Vehicle			0.0735	0.1456	0.1126	0.9424	0.2073	0.7873

Total Cumulative Damage:

$$\begin{aligned}\Sigma v_i \phi_i^3 &= \Sigma [(\text{Percentage by Class of Vehicle}) (\Sigma v_i \phi_i^3 \text{ by Class of Vehicle})] \\ &= 0.0735*0.0164+0.1456*0.0026+0.1126*0.0058+0.9424*0.0223+0.2073*0.0273+0.7873*0.9256 \\ &= 0.76\end{aligned}$$

$$\text{PTF} = (0.76)^{1/3}/0.75 = 1.22, \text{ Use PTF } 1.2$$

5.1.1.1.3 Design Fatigue Life (in Cycles)

The number of cycles to failure (fatigue life, based on a probability of failure of 2.3%) can be determined from

$$N_{\text{design}} = A [\gamma (\text{PTF}) (\Delta f)_e]^{-3} \text{ or } A [\gamma (\Delta f)_e]^{-3} \quad (5.1.1.1.3-1)$$

where:

N_{design} = estimated minimum number of cycles to failure

$\gamma(\text{PTF})(\Delta f)_e$ = calculated effective stress range, based on Equation PP5.1.1.1.2b-2 (ksi)

A = constant (values given in Table A6.6.1.2.5-1)

$\gamma(\Delta f)_e$ = calculated effective stress range, based on Equation PP5.1.1.1.2b-3 (ksi)

PTF = Pennsylvania Traffic Factor as given in D6.6.1.2.2

The probability of failure associated with the LRFD design fatigue life is 2.3%.

These equations are plotted in Figure AC6.6.1.2.5-1, along with the constant amplitude fatigue thresholds. The design equations were developed by observing laboratory fatigue failures of various details. Typically, fatigue failures fall within a wide scatterband. This phenomenon can be observed in Figure 5.1.1.1.3-1. In this figure are plotted the observed laboratory fatigue failures of welded beams without any welded attachments, used in the development of Category B. Also shown is a solid line through the data points which represents the mean, on the basis of the principle of least squares. A distance of two standard errors of estimate (in this case, the standard error of estimate approximates the standard deviation) from either side of this mean line is shown as parallel dashed lines; these lines form an envelope which statistically should contain about 95% of all Category B fatigue failures. About 2.3% of all Category B fatigue failures can be expected to occur at a number of cycles greater than that indicated by the envelope. Analogously, about 2.3% can be expected at a number of cycles less than that indicated by the envelope. The lower bound of this “95% confidence limit” envelope has been used as the Category B LRFD design curve. Using this lower bound of the wide scatterband as a design equation is reasonable and yields an adequate margin of safety against fatigue failures.

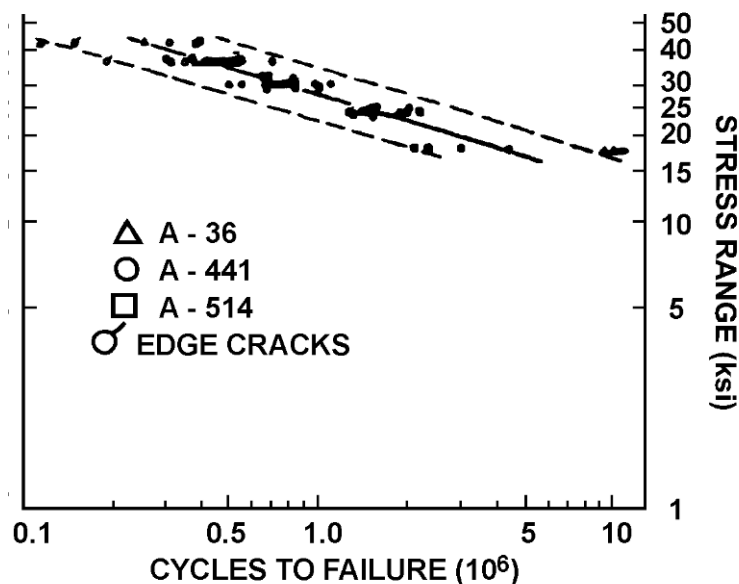


Figure 5.1.1.1.3-1 – Laboratory Fatigue Failures of Welded Beams

Using the lower bound to estimate remaining fatigue life in an evaluation context is another matter. The large majority of Category B details will exhibit fatigue lives in excess of those predicted by the design fatigue life equation using Table A6.6.1.2.5-1. For example, at an effective stress range of 20 ksi, the design equation derived for Category B predicts a fatigue life of 1.5 million cycles. This calculated value of fatigue life, however, is a lower bound. At the other end of the 95% confidence limit envelope, the calculated fatigue life is just under 6 million cycles. Thus, it can be expected that 95% of all Category B details cycled at a stress range of 20 ksi will fail between 1.5 and 6 million cycles. The width of the scatterband of resultant fatigue life is very large.

The probability of failure associated with this design fatigue life is about 2.3%, or 1 in 43.5. This means that 2.3% of the test data fall on each side of the so-called 95% confidence limits. In other words, only 2.3% of all details would be expected to fail prior to this calculated life. Another 2.3% would be expected to fail after the higher fatigue life indicated by the upper 95% confidence limit.

In summary, the design fatigue life equations, represented by Equation 5.1.1.1.3-1 with the use of Table A6.6.1.2.5-1, can be used to estimate the fatigue life of welded, bolted or riveted details. They provide lower bounds on life. For non-redundant or less redundant load path structures, where cracking can have dire consequences, it may be prudent to use these equations to predict safe life. However, for more redundant structures, these equations may prove too conservative for practical purposes.

5.1.1.1.4 Evaluation Fatigue Life

When the design fatigue life equations (with the use of Table A6.6.1.2.5-1) prove to be too conservative (which will probably be the case more often than not), Table 5.1.1.1.4-1 may be used to convert calculated design fatigue life into an

expected fatigue life with a greater (more realistic) probability of failure, for use in evaluation. To determine the evaluation fatigue life, the design fatigue life calculated according to PP5.1.1.1.3, shall be multiplied by the multiplier in Table 5.1.1.1.4-1 corresponding to an acceptable probability of failure.

The probability of failure relates to the initiation of cracking at a welded, bolted, or riveted detail. This probability should not be confused with that of the collapse of the bridge. Cracking at a fatigue-sensitive detail of a non-redundant cross-section (e.g., the tie of a tied-arch) could certainly precipitate rapid collapse of the bridge, but failure of one Category E' cover plate end welded detail in a multi-girder bridge is not necessarily tantamount to the collapse of that bridge. Thus, the acceptable probability of failure should be a function of the bridge's redundancy.

The period when the bridge was constructed may also play a role in the assignment of an acceptable probability of failure. Bridges fabricated prior to the adoption of AASHTO's Fracture Control Plan (1978) may have lower fracture toughness levels than are currently deemed acceptable. Without destructive material testing to ascertain toughness levels, the acceptance of a probability of failure greater than that used for design (greater than 2.3%) is questionable. An even lower value than this may be appropriate.

If there are poor details (i.e., those explicitly excluded from current design practice), such as intersecting fillet welds, a lower acceptable probability of failure should be selected.

Final approval of the acceptable probability of failure is the responsibility of the Chief Bridge Engineer.

Table 5.1.1.1.4-1 – Multiplier to Convert Design Fatigue Life to Evaluation Fatigue Life

Probability of Failure							
Category	0.5%	1.0%	2.3%	5%	10%	25%	50%
A	0.75	0.85	1.00	1.20	1.44	1.96	2.77
B	0.82	0.89	1.00	1.13	1.27	1.57	1.97
B'	0.78	0.87	1.00	1.16	1.36	1.76	2.37
C & C'	0.92	0.95	1.00	1.05	1.11	1.21	1.34
D	0.87	0.93	1.00	1.09	1.18	1.36	1.59
E	0.87	0.93	1.00	1.09	1.18	1.36	1.59
E'	0.76	0.86	1.00	1.18	1.39	1.84	2.51

If the Engineer wishes to assign an acceptable probability of failure other than those shown in Table 5.1.1.1.4-1, the following general equation can be used to assess accumulated load-induced fatigue damage for use in evaluation:

$$N = CD^{-n} [\gamma(PTF)(\Delta f)_e]^{-3} \quad \text{or} \quad CD^{-1} [\gamma(\Delta f)_e]^{-3} \quad (5.1.1.1.4-1)$$

where:

N = estimated number of cycles to failure

C = fatigue life coefficient as given in Table 5.1.1.1.4-2

D = probability of failure coefficient as given in Table 5.1.1.1.4-2

n = probability of failure exponent as given in Table 5.1.1.1.4-3

$\gamma(PTF)(\Delta f)_e$ = effective stress range based on Equation PP5.1.1.1.2b-2

$\gamma(\Delta f)_e$ = effective stress range based on Equation PP5.1.1.1.2b-3

PTF = Pennsylvania Traffic Factor as given in D6.6.1.2.2

Table 5.1.1.1.4-2 – Values for General Fatigue Equations

Category I	Constant C	Constant D
A	2.27×10^{13}	1.66
B	7.67×10^{12}	1.40
B'	4.72×10^{12}	1.54
C & C'	1.95×10^{12}	1.16
D	1.13×10^{12}	1.26
E	5.61×10^{11}	1.26
E'	3.22×10^{11}	1.59

Table 5.1.1.1.4-3 – Probability of Failure

Probability of Failure (%)	Exponent
98.0%	-2.0
75.0%	-0.675
50.0%	0.0
25.0%	0.675
10.0%	1.285
5.0%	1.645
2.3%	2.0
1.0%	2.327
.5%	2.576
.25%	2.810
.1%	3.090

All of the above estimates represent total life. The residual life of an existing detail is equal to this total life minus its life to-date.

5.1.1.1.5 Example Fatigue Damage Assessment

The following threefold example of fatigue damage assessment is taken in part from a Department study of the Clarion River Bridge along I-80. At the time of the study, the bridge was in service for about 15 years.

The example is based on using a fatigue stress range, $(\Delta f)_D$ at a Category E' cover plate detail on a longitudinal member of 1.93 ksi. The average past ADTT to the time of the assessment was determined from Department data to be 3,900 trucks per day, and based on Department projections, a reasonable average future ADTT for the bridge is 8,650 trucks per day. The current ADTT was 5,000 trucks per day.

5.1.1.1.5a Simplified Damage Assessment

The simplified fatigue damage assessment is based upon the use of Equation PP5.1.1.1.2b-2. The effective stress range can be estimated from the calculated fatigue stress range using Equation PP5.1.1.1.2b-2:

$$\begin{aligned}\gamma(\text{PTF})(\Delta f)_e &= \gamma(\text{PTF}) \alpha (\Delta f)_D \\ &= 0.75(1.2)(1.0)(1.93 \text{ ksi}) \\ &= 1.74 \text{ ksi}\end{aligned}$$

The fatigue life shall be estimated using Equation PP5.1.1.1.3-1

$$\begin{aligned}N_{\text{design}} &= A[\gamma(\text{PTF})(\Delta f)_e]^{-3} \\ &= (3.9 \times 10^8)[1.74 \text{ ksi}]^{-3} \\ &= 74.0 \times 10^6 \text{ cycles}\end{aligned}$$

Up to the present, the structural detail has experienced 21.4×10^6 cycles (assuming one cycle per truck, 3,900 trucks per day for 15 years). Thus, significant residual life remains about 53 million cycles, or, at the future ADTT of 8,650 trucks per day, about 17 years.

If a less conservative probability of failure than that in to the LRFD design provisions is justified, the multiplier in Table PP5.1.1.1.4-1 can be used. If a probability of failure as high as 50% could be justified due to multiple redundancy, a multiplier of 2.51 could be applied to the design fatigue life:

$$\begin{aligned}N_{\text{evaluation}} &= 2.51 N_{\text{design}} \\ &= 2.51 (74.0 \times 10^6 \text{ cycles}) \\ &= 186 \times 10^6 \text{ cycles}\end{aligned}$$

Now the residual life is about 164 million cycles or, at the projected future ADTT of 8,650 trucks per day, 52 years.

5.1.1.1.5b Assessment Using Traffic Study

Traffic studies can be used to add a degree of refinement to the fatigue damage assessment. While the most readily implementable data is GVW distribution, the acquisition of such data would require weigh stations specific to the bridge site. Site-specific distributions of truck traffic by truck type are more readily obtainable than distributions of traffic by gross vehicle weight. Generation of distributions by type requires mere observation and counting of trucks; determining distribution by weight requires determining the actual weight of the trucks.

If the assumption is made that the distribution of gross vehicle weights within each truck type (as indicated by statewide weighing) is constant, the cumulative damages by truck type can be proportioned on the basis of the observed distribution by truck type to obtain a site-specific GVW distribution.

On the basis of the sampling of 5,923 significant vehicles (i.e. those over 20 kips), shown in Table 5.1.1.1.5b-1, the 1973 statewide break-down of traffic by truck type can be assumed to be as follows:

<u>Truck Type</u>		<u>Traffic (%)</u>
Single Unit Trucks	- 2 axles	3.8
	- 3 axles	3.1
Tractor Semi-Trailer Combinations	- 3 axles	2.6
	- 4 axles	16.8
	- 5 axles or more	73.6

Using this same sampling, the cumulative damage ($\Sigma v_i \phi_i^3$) by truck type can be calculated to be as follows:

<u>Truck Type</u>		<u>$\Sigma v_i \phi_i^3$</u>
Single Unit Trucks	- 2 axles	0.0530
	- 3 axles	0.19241
Tractor Semi-Trailer Combinations	- 3 axles	0.08412
	- 4 axles	0.21074
	- 5 axles or more	0.60261

Combining these summations of cumulative damage with the percentages of truck types above yields the following equation:

$$\Sigma v_i \phi_i^3 = 0.038(0.05304) + 0.031(0.19241) + 0.026(0.08412) + 0.168(0.21074) + 0.736(0.60261) = 0.489$$

The foregoing calculation led to the value of $\Sigma v_i \phi_i^3$ of 0.50, which was used before the 1993 data was available.

The breakdown of cumulative damage by truck type, as shown above, allows for improvements in estimated damage where different distributions of traffic by truck type are encountered. For the preceding example, a site-specific distribution by truck type was also available, as shown in Table 5.1.1.1.5b-1. More reliable estimates for fatigue damage are obtained by utilizing the site-specific data. Combining the statewide summations of cumulative damage by site-specific percentages of truck types yields the following estimate of total damage:

$$\Sigma v_i \phi_i^3 = \Sigma [(\text{Percentage of truck type from Table 5.1.1.1.5b-1}) (\Sigma v_i \phi_i^3 \text{ by truck type from Table PP5.1.1.1.2b-1})]$$

$$\Sigma v_i \phi_i^3 = 0.065(0.05304) + 0.017(0.19241) + 0.014(0.08412) + 0.083(0.21074) + 0.822(0.60261) = 0.521$$

This example of the Clarion River Bridge along I-80 (based on 1982 data) suggests that the cumulative damage along I-80 may be less than that suggested by the 1993 GVW survey.

This more accurate value can now be used in Equation PP5.1.1.1.2b-3 in conjunction with a value of α consistent with the LRFD design provisions.

$$\begin{aligned} \gamma (\Delta f)_e &= [\Sigma v_i \phi_i^3]^{1/3} \alpha (\Delta f)_D \\ &= [0.521]^{1/3} (1.0) 1.93 \text{ ksi} \\ &= 1.55 \text{ ksi} \end{aligned}$$

The 1.55 ksi compares with 1.74 ksi using the simplified approach (see PP5.1.1.1.5a).

In this case, the results of the assessment using a traffic study will not be significantly different from the simplified approach, but more faith can be placed in the assessment because of the use of the site-specific traffic study.

Table 5.1.1.1.5b-1 – Clarion River Bridge Truck Type Distribution

Year	ADTT	Single-Unit Trucks				Tractor Semi-Trailer Combinations					
		2 Axle		3 Axle		3 Axle		4 Axle		5 Axle or More	
		%	#	%	#	%	#	%	#	%	#
68	350	14	49	4	14	5	18	42	147	35	123
69	550	14	77	4	22	5	28	38	209	39	215
70	1750	10	175	3	53	3	53	27	473	57	998
71	3800	6	228	2	76	2	76	13	494	77	2926
72	3900	7	273	3	117	2	78	11	429	78	3042
73	3950	6	237	1	40	1	40	10	395	82	3239
74	4450	6	267	1	45	1	45	10	445	82	3649
75	4550	6	273	1	46	2	91	8	364	83	3777
76	4800	6	288	1	48	2	96	7	336	84	4032
77	5000	5	250	1	50	1	50	7	350	86	4300
78	5200	5	260	1	52	1	52	6	312	87	4524
79	5200	5	260	2	104	1	52	5	260	87	4524
80	5100	5	255	2	102	1	51	5	255	87	4437
81	5150	8	412	2	103	1	52	4	206	85	4378
82	4950	10	495	3	149	1	50	4	198	82	4059
	58 700		3799		1021		832		4873		48 223
Weighed Average %			6.5%		1.7%		1.4%		8.3%		82.2%

5.1.1.1.5c Assessment Using Measured Stresses

The stress range spectrum can be estimated by measuring strains at critical structural details over an extended period of time to capture a representative sample of truck traffic. In this case, the observed effective stress range, $\gamma(\Delta f)_e$, was just under 1.0 ksi, while the observed maximum stress range during the period of observation was just under 3.0 ksi.

Since the maximum stress range of 3.0 ksi is greater than infinite life threshold value of 2.6 ksi for Category E' detail from Table A6.6.1.2.5-3, the estimated design fatigue life using Equation PP5.1.1.1.3-1 results in:

$$\begin{aligned}
 N_{\text{design}} &= 3.9 \times 10^8 (1.0)^{-3} \\
 &= 390 \times 10^6 \text{ cycles}
 \end{aligned}$$

Even at the low probability of failure associated with the LRFD design provisions (2.3%), the calculated residual life of about 370 million cycles (or 117 years at 8,650 trucks per day) is in excess of any specified life for the bridge. Varying the probability of failure to determine an evaluation fatigue life is not necessary in this case.

5.1.1.1.5d Effect of Permit Vehicles

To illustrate the potential effect of permit vehicles on accumulated fatigue damage, the following example is presented. Gross vehicle weight recorded for all the overweight permits issued by the Department for one calendar year were reduced to a value of the summation of $v_i \phi_i^3$ equal to 3.21 (see Table 5.1.1.1.5d-1) (as compared with 0.76 based on the 1993 GVW surveys, as stated earlier). In the 1993 GVW distribution, approximately 14% of the vehicles exceeded 80.2 kips. This figure represents the vehicles observed during a particular period, potentially including any permit vehicles. If it is assumed that 4% of all trucks in the through lane are permit vehicles, and they are added to the observed distribution which already includes 14% of the vehicles over 80.2 kips, then the combined summation of $v_i \phi_i^3$ is equal to

$$(0.96)(0.76) + (0.04)(3.21) = 0.86$$

Using this value in the example of a simplified assessment in PP5.1.1.1.5a yields

$$\begin{aligned}\gamma (\Delta f)_e &= [\sum v_i \phi_i^3]^{1/3} \alpha (\Delta f)_D \\ &= [0.86]^{1/3} (1.0) 1.93 \text{ ksi} \\ &= 1.84 \text{ ksi}\end{aligned}$$

as opposed to 1.74 ksi without permit vehicles included. Now,

$$\begin{aligned}N_{\text{design}} &= 3.9 \times 10^8 (1.84)^{-3} \\ &= 63.1 \times 10^6 \text{ cycles}\end{aligned}$$

as opposed to about 74 million cycles when permit vehicles are excluded. In other words, when permit vehicles are factored into the analysis as 4% of the truck traffic distribution, the number of anticipated cycles to cracking is reduced to about 85% of that when permit vehicles are neglected.

Table 5.1.1.1.5d-1 – Permit Load Cumulative Damage

Weight (kips)	ϕ_i	#	v_i	$v_i \phi_i^3$
73.281 - 95.0	1.169	24,547	0.4744	0.7579
95.1 - 123.0	1.514	18,989	0.3670	1.2736
123.1 - 150.0	1.896	7,557	0.1460	0.9951
150.1 - 177.0	2.271	435	0.0035	0.0648
177.1 - 204.0	2.646	179	0.0035	0.0648
over 204.0	2.833	39	0.0008	0.0182
		51,746	1.0000	3.2080

5.1.2 Displacement-Induced Fatigue

Displacement-induced fatigue is usually associated with relatively small out-of-plane displacements. In many cases, local relative movements measured in only thousandths of an inch (mils) have been sufficient to cause displacement-induced fatigue damage. Such displacement-induced fatigue cracking is epidemic on steel bridges throughout America.

Two conditions are necessary for displacement-induced fatigue damage to occur:

- (a) A periodic out-of-plane force or displacement.
- (b) An abrupt local change in stiffness where the force/displacement is applied.

Without the combination of stiffness changes and driving force, displacement-induced fatigue cracking will not develop. The presence of one of the above-mentioned conditions is not in itself sufficient to cause displacement-induced cracking. For example, the end of a transverse stiffener (not a transverse connection plate) of a plate girder cut short of the tension flange (for load-induced fatigue considerations) constitutes an abrupt change in stiffness in the out-of-plane direction of the plate girder web. After erection and placement into service, no out-of-plane force exists to oppose the change in out-of-plane stiffness. The potential for in-service displacement-induced cracking at the cut-short stiffener end does not exist, since both conditions (stiffness change and driving force) are not present. In some documented cases, cracks have been observed at cut-short ends of transverse stiffeners before the girder was put into service. It has been shown that such cracking was the result of inadequate blocking for shipment by rail. The periodic rocking of the railway car induced an inertial force of the tension flange in opposition to the stiffness change. In this case, both conditions were present and the potential for cracking was realized.

5.1.2.1 Displacement-Induced Fatigue Damage Assessment

Cumulative fatigue damage of members and fasteners subject to repeated variations or reversals of stress due to out-of-plane deformations or secondary forces shall not be assessed.

The time-honored simplifying assumptions made to analyze bridges does not provide the tools necessary to quantify these out-of-plane displacements or the stresses that result from them. In the past, much of the effort in bridge research dealt with developing simplified tools by which a complex three-dimensional structure could be analyzed as a simple one-dimensional element. For example, in the design process for a simple multi-girder bridge, this highly interconnected assemblage of deck, girders, cross-frames and laterals is idealized as a single line element and analyzed as an isolated beam. Any ability to calculate forces or displacements normal to the plane of the idealized line element is lost.

Refined methods of analysis enable designers to take some of the three-dimensionality of the bridge into account. For example, in the multi-girder type of bridge discussed above, it is now practical to use grillage or finite element methods to obtain very realistic load distribution among the several girders and to quantify forces in laterals and cross-frames. As large an improvement as this is, it still does not necessarily lead to practical design-office evaluation of displacement-induced stresses because an entirely different order of magnitude in element sizes (hence in numbers of nodes and elements) is required to calculate these very localized stresses.

5.1.3 Fracture-Critical Cross Girder Pier Caps Cracking at Internal Diaphragms

Box girder pier caps are classified as fracture-critical. The connection detail of the internal diaphragm of a box girder pier cap which provides continuity through nested steel beams is highly susceptible to fatigue cracking unless the diaphragms are connected to both flanges and webs of the box girder.

An NBIS inspection on a steel box cross girder pier cap with a multi-girder superstructure framing into it revealed that the box girder's interior diaphragm was welded on three sides, but did not connect to the bottom tension flange as shown in Figure 5.1.3-1. A "U-shaped" crack in the web at the bottom of one diaphragm was detected visually and then was verified with dye-penetrant. Unchecked, the crack could have extended into the flanges and caused the eleven year old girder to fail.

The detail of providing only a partial depth internal diaphragm, as shown in Figure 5.1.3-1, was formerly believed to avoid problems associated with welding tension flanges. However, this gap allows out-of-plane bending and creates possibility of web cracking at the end of the partial depth diaphragm near both tension and compression flanges. This cracking may extend completely through the web or only partially if lamellar tearing also occurs. Because these cross-girders are fracture-critical, appropriate measures must be taken to avoid a sudden failure and to retrofit the girder for continued use. Do not use this type of detail on future projects.

Retrofit this type of detail as follows:

1. Connect the diaphragm to the flange with bolted steel angles (see Figure 5.1.3-2).
2. Provide friction connection with high strength bolts.
3. Bolts placed at maximum pitch will suffice.
4. Because the new bolt holes will reduce the cross-sectional area of the tension flanges, the capacity of the cross girder must be checked. Maintain a minimum strength of PHL-93 inventory and P-82 permit loading with LRFD method. Stagger bolts on tension flanges as necessary to maintain girder strength.

If the above retrofit cannot be performed in a timely manner, and if cracking is observed, an interim retrofit as shown in Figure 5.1.3-3 would suffice. This was used in District 6-0, and has performed well since 1986.

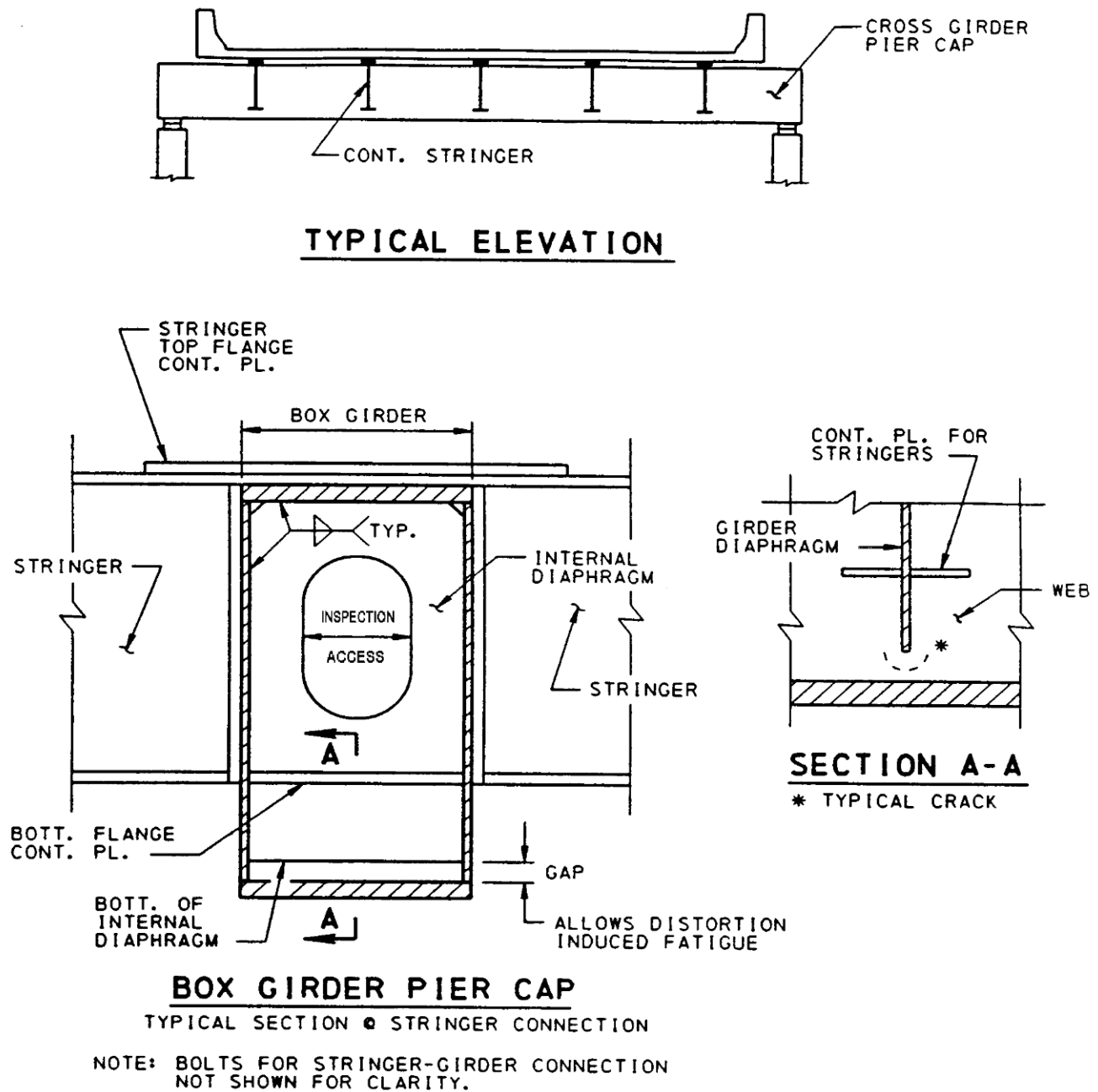
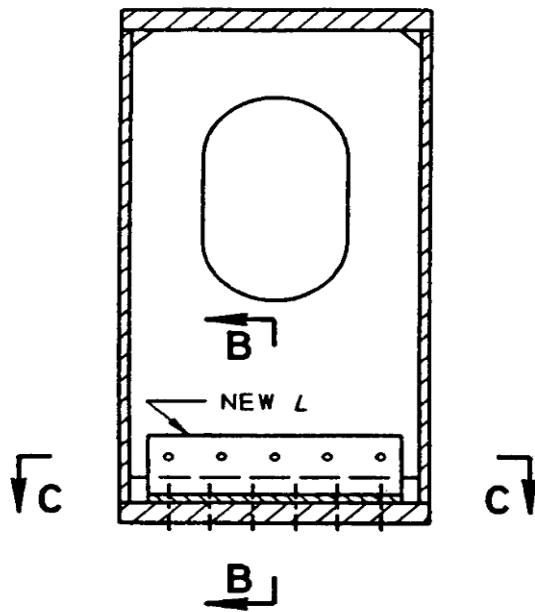
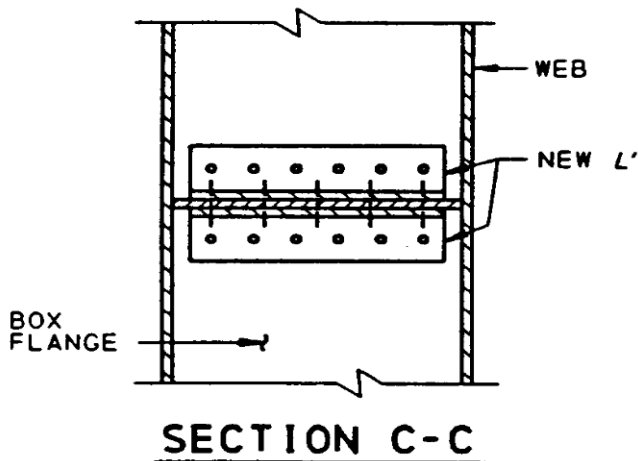


Figure 5.1.3-1 – Fracture-Critical Girder Pier Caps

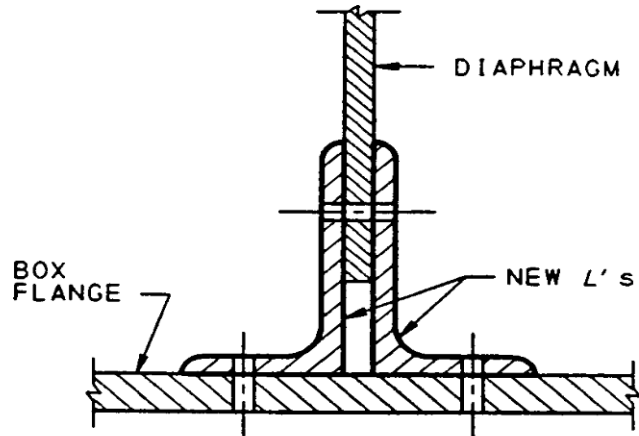


TYPICAL SECTION

STRINGER DETAILS OMITTED
FOR CLARITY



SECTION C-C



SECTION B-B

NOTES:

1. USE H. S. BOLTS.
2. PROVIDE FRICTION CONNECTION.
3. MAINTAIN PHL - 93 @ I.R.
AND P - 82 @ O.R. MIN.
4. CONSIDER STAGGERING BOLTS IN
FLANGE TO MINIMIZE LOSS OF
X-SECTION AREA FOR TENSION
FLANGES.



5. DETAILS SHOWN ARE TYPICAL FOR
TENSION FLANGES. PROVIDE SAME
FOR COMPRESSION FLANGE IF
NO POSITIVE CONNECTION EXISTS.

Figure 5.1.3-2 – Preferred Retrofit Details

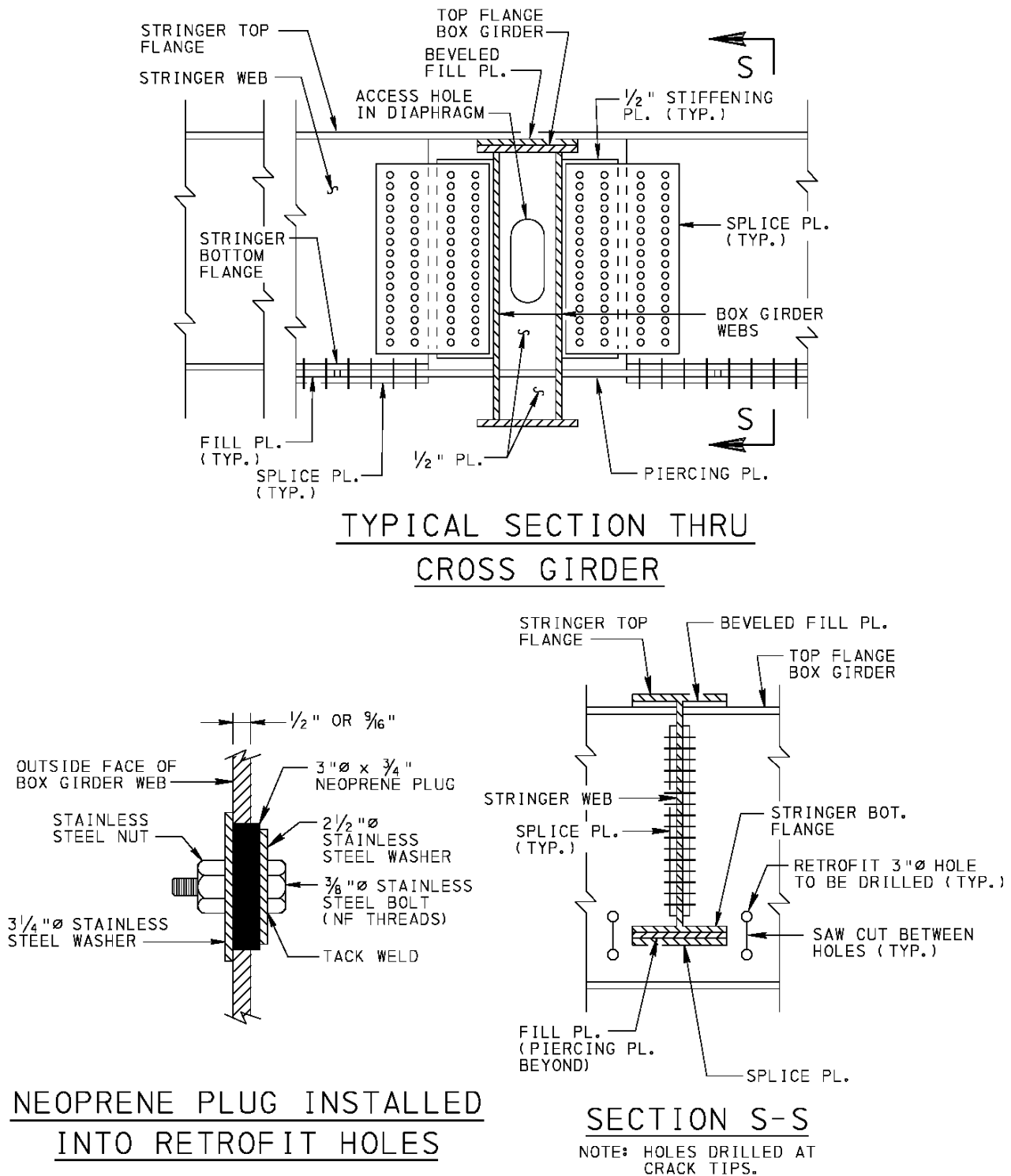


Figure 5.1.3-3 – Alternate Retrofit Details

5.2 COVER PLATE CUTOFFS

The articles in this section are intended for rehabilitation projects in which the use of partial-length cover plates may be necessary.

5.2.1 General

Any cover plate that is not required to be full-length shall extend beyond the theoretical cutoff point far enough to develop the capacity of the plate or shall extend beyond a section where the stress in the remainder of the girder flange is equal to the nominal fatigue resistance, whichever is greater. The theoretical cutoff point of the cover plate is the section at which the stress in the flange without the cover plate equals the flange resistance, exclusive of fatigue considerations.

5.2.1.1 Welded Cover Plates

Any partial-length welded cover plate shall extend beyond the theoretical end by the terminal distance, or it shall extend beyond a section where the stress range in the beam flange without the cover plate is equal to the nominal fatigue resistance for base metal adjacent to or connected to by fillet welds. The theoretical end of the cover plate is the section at which the stress in the flange without the cover plate equals the flange resistance, exclusive of fatigue considerations. The terminal distance is two times the nominal cover plate width for cover plates not welded across their ends and one and one-half times for cover plates welded across their ends. The width at ends of tapered cover plates shall not be less than 3 in.. The weld connecting the cover plate shall be of sufficient size to develop a total stress of not less than the computed stress in the cover plate at its theoretical end.

5.2.1.2 Retrofitting of Existing Welded Cover Plates

Rolled beam steel girders with welded partial length cover plates have experienced fatigue cracking at the end of cover plates. Research shows that end bolting of the cover plates appreciably increases the fatigue life at the end of cover plates. This is very significant in case of retrofitting partial length cover plated beams. New cover plated rolled beams shall be designed in accordance with A6.10.12 and D6.10.12.

Whenever retrofitting is required on existing bridges, the cover plates must be end bolted as detailed in Figure 5.2.1.2-1. The number and size of high strength bolts may vary. The design procedure is as follows:

1. For non-cracked flanges at cover plate ends:

Design Moment M

- a. Calculate area of bolt holes, Δ_A , in excess of 15% of gross flange area.
- b. Deduct Δ_A from gross flange area.
- c. Calculate net moment of inertia of beam with bolt holes, I_{net} .

$$I_{net} = I - \Delta_A \left(\frac{d}{2} - \frac{t_f}{2} \right)^2$$

- d. Calculate, $S_{net} = \frac{I_{net}}{\frac{d}{2}}$
- e. Calculate design moment, $M = S_{net} F_y$

Moment Carried by Web

$$\text{Stress at extreme fiber of web, } f = \frac{F_y \left(\frac{d}{2} - t_f \right)}{\frac{d}{2}}$$

$$\text{Section modulus of web, } S_w = \frac{1}{6} t_w (d - 2t_f)^2$$

$$\text{Moment carried by web, } M_w = S_w f$$

$$\text{Force carried by flange, } T = \frac{M_f}{d}$$

Required Number of Bolts

The force carried by a bolt shall not exceed the nominal slip resistance (R_n) specified in A6.13.2.8 and D6.13.2.8:

$$\text{Number of bolts required with two slip planes} = \frac{T}{R_n} \text{ (Round to the next even number)}$$

Required Splice Plate Area

$$A_{\text{reqd}} = \frac{T}{F_y}$$

Select the plate size so that $*A_{\text{reqd}}$ = gross area minus area of bolt holes in excess of 15% of gross area.

* A_{reqd} should not be less than the flange area.

2. For cracked flanges at the cover plate ends, design the splice plates for the design moment, M.

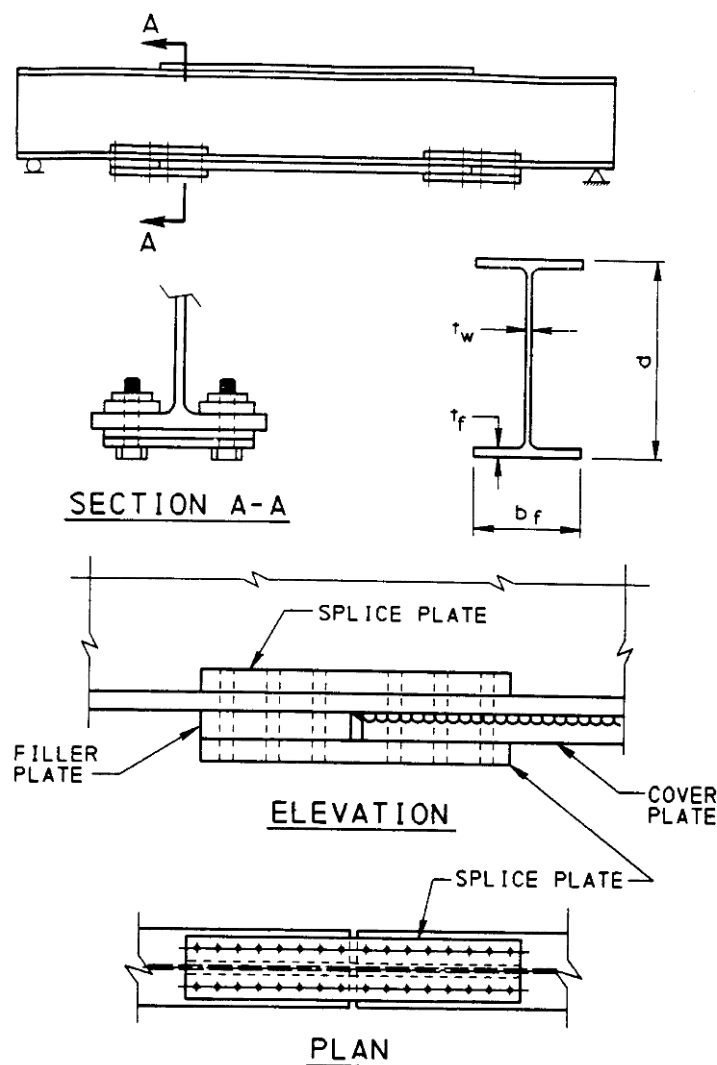


Figure 5.2.1.2-1 – Partial Length Cover Plates

The following is an example of above-outlined bolt design procedure.

Assumed Beam size=W36 x 160, flange cracked and A 36 steel

Area of flange $A_f = b_f t_f = 12.000 \times 1.020 = 12.240 \text{ in}^2$

Area of two bolt holes in the flange $= 2t_f d_h = 2 \times 1.020 \times 1.125 = 2.295 \text{ in}^2$

Assuming 1 in. dia. (A325) bolts, d_h = hole diameter, 1/8" larger than bolt diameter.

Area of bolt holes in excess of 15% of flange area, $\Delta_A = 2t_f d_h - 0.15A_f$
 $= 2.295 - (0.15 \times 12.240) = 0.459 \text{ in}^2$

Therefore, 0.459 in^2 is to be deducted from the gross area for calculation of stresses.

$$I_{\text{net}} = I - \Delta_A \left(\frac{d}{2} - \frac{t_f}{2} \right)^2 = 9750 - 0.459 \left(\frac{36.01}{2} - \frac{1.020}{2} \right)^2 = 9609.5 \text{ in}^4$$

$$S_{\text{net}} = \frac{I_{\text{net}}}{\left(\frac{d}{2}\right)} = \left(\frac{9609.5}{\frac{36.01}{2}}\right) = 533.7 \text{ in}^3$$

Design stress $F_y = 36 \text{ ksi}$

Therefore, Design moment = $S_{\text{net}} F_y = 533.7 \times 36 = 19.2 \times 10^3 \text{ k}\cdot\text{in}$

Calculate moment carried by web:

$$\text{Stress at extreme fiber of web, } f = F_y \frac{\left(\frac{d}{2} - t_f\right)}{\left(\frac{d}{2}\right)} = 36 \times \frac{\left(\frac{36.01}{2} - 1.020\right)}{\left(\frac{36.01}{2}\right)} = 33.96 \text{ ksi}$$

Section modulus of web $S_w = \frac{1}{6} t_w (d - 2t_f)^2$

$$S_w = (1/6) \times 0.650 \times (36.01 - 2 \times 1.020)^2 = 125 \text{ in}^3$$

Moment carried by web = $S_w f = 125 \times 33.96 = 4.25 \times 10^3 \text{ k}\cdot\text{in}$

Moment carried by flange, $M_f = 19.2 \times 10^3 - 4.25 \times 10^3 = 14.95 \times 10^3 \text{ k}\cdot\text{in}$

$$\text{Force carried by flange, } T = \frac{M_f}{d} = \frac{14.95 \times 10^3}{36.01} = 416 \text{ kips}$$

Required number of bolts:

$$R_n = K_h K_s N_s P_t$$

$K_h = 1.0$ (for standard holes from Table A6.13.2.8-2)

$K_s = 0.33$ (for Class A surface conditions from Table A6.13.2.8-3)

$N_s = 2$ (number of slip planes)

$P_t = 51 \text{ kips}$ (for 1 in. dia. bolt from Table A6.13.2.8-1)

Therefore, $R_n = 1.0 \times 0.33 \times 2 \times 51 = 33.7 \text{ kips}$

$N_b = \text{number of bolts in the joint}$

$$\text{or } N_b = \frac{T}{R_n} = \frac{416}{33.7} = 12.34$$

USE: 14 - 1 in. dia. A325 HS bolts

Required Splice Plate Area

$$A_{\text{req'd}} = \frac{T}{F_y} = \frac{416}{36} = 11.56 \text{ in}^2$$

Assuming an outer plate $10\frac{1}{2}$ " x $\frac{3}{4}$ " and two inside plates 4" x $\frac{3}{4}$ "

$$A_{\text{provided}} = (10.50 \times 0.75) + 2(4.00 \times 0.75) = 13.875 \text{ in}^2$$

$$15\% \text{ of } A_{\text{provided}} = 13.875 \times 0.15 = 2.081 \text{ in}^2$$

$$\text{Area of four holes in splice plates for 1 in. dia. bolts} = 4t d_h = 4 \times 0.75 \times 1.125 = 3.375 \text{ in}^2$$

$$\text{Area to be deducted from gross area} = 3.375 - 2.081 = 1.294 \text{ in}^2$$

$$A_{\text{net}} = 13.875 - 1.294 = 12.58 \text{ in}^2 > 11.56 \text{ in}^2$$

The retrofit shall be designed for fatigue using the constant amplitude fatigue thresholds of LRFD Category B, regardless of whether the flange is non-cracked or cracked.

The tip of any crack that has entered the web shall be arrested by drilling a hole, cleaning the area, installing a high-strength bolt of the same diameter as is used for the splice, and torquing it to the prescribed initial tension. Extra care shall be taken that the hole does not miss the crack tip.

If the web crack is longer than one sixth of the beam depth, the web shall be considered fully cracked. Therefore, the splice shall be designed as a full flange and web splice using A6.13 and D6.13.

5.3 FASTENERS

For rehabilitation of riveted members carrying calculated stress, 5/8 in. diameter fasteners shall be used only if 3/4 in. diameter or larger fasteners will not fit.

5.4 WELD REPAIRS OF TENSION FLANGES OF A-7 STEEL BRIDGES

Sometimes numerous nicks and cuts are inflicted on the top flanges of beams of simple and continuous girder spans when the contractors saw-cut the bridge deck transversely to facilitate its removal. Saw-cut may also go through the rivet heads and edges of flanges. Some damages can be inflicted by blows from a jackhammer. Consequently, it becomes necessary to establish the weldability of the steel by performing a chemical analysis and determining the carbon equivalent of the material, in order to approve the method of weld repairs, especially for the large saw cuts in the tension area of the top flanges of the girders, which are considered as critical members that are subject to fatigue consideration due to the creation of various geometrical notches. Welding on the tension flange should be avoided whenever possible. All weld repairs shall be treated with Ultrasonic Impact Treatment (UIT) process. Weld repairs should be considered only for major cuts otherwise nicks and cuts should be ground to bright metal in the direction of stress for both tension flanges, compression flanges or webs. The excavation shall be smooth and free of irregularities. Material shall be faired with a slope not to exceed one in ten. It may be necessary to verify the member has adequate cross sectional area due to the removal of the nicks and gouges.

Use the following criteria when welding in the field becomes necessary to repair damages inflicted to critical members of A-7 steel bridges in field operations:

1. Perform the chemical analysis of a plate to determine the feasibility of welding.

The current specifications for A 36/A 36M, A 242/A 242M, Type 2, A 572/A 572M and A500 steels limit the sulfur and phosphorus to a maximum of 0.05% and 0.04%, respectively. If the chemical analysis of a plate reveals higher sulfur and/or phosphorus, do not perform welding. Phosphorus and sulfur are undesirable impurities which embrittle the steel and weld metal.

With regard to cold cracking, based on the expression

$$C + M_n/6 = CE$$

The carbon equivalent (CE) must not exceed 0.40%. For steel with CE equal to or less than 0.40, use ANSI/AASHTO/AWS D1.5 Chapter 12 including preheat Section 12.17.6(8)(a) and post heat Section 12.15. If the CE of any plate to be weld repaired exceeds 0.40, then particular attention must be given to low-hydrogen practice and the time between welding and inspection (per Section 12.16.4) should be increased to 48 hours for all plates. The welds should be ground flush before inspection. After grinding and prior to RT/UT, the welds should be dye-penetrant (DT) inspected. In UT, the scanning should include pattern D as, well as pattern E (ANSI/AASHTO/AWS D1.5), with UT from both top and bottom surfaces of the flanges.

2. Place proper documentation of the damages, such as location, type, size, stress range at damaged location, method of weld repair, method of N.D.T., etc., in the District's Structure Inventory and Appraisal (inspection) file.
3. Treat weld repair areas with Ultrasonic Impact Treatment process

5.5 BRIDGE REHABILITATION STRATEGIES

The items included herein are not all inclusive. Depending upon the type of rehabilitation needs, some or all of the items may be applicable.

In evaluating the rehabilitation or replacement of any component of a bridge, every component, as well as structural capacity, deck geometry, scour, seismic adequacy, and other deficiencies need to be assessed.

5.5.1 Needs Establishment

Rehabilitation of an existing bridge may be warranted by its condition, special situation, or by inclusion on a program, such as the Interstate Pavement Preservation (PPG), highway capacity improvement, safety improvement or other structural improvement program.

A Pennsylvania condition rating (refer to Publication 100A, Bridge Management System 2 (BMS2), Coding Manual) of six or less for the entire bridge or its components would indicate the need for rehabilitation.

If the sufficiency rating (SR) of a bridge is less than 50.0 and the bridge is structurally deficient or functionally obsolete, it is eligible for Highway Bridge Program (HBP) funds for rehabilitation or replacement. If the SR is greater than or equal to 50.0, but less than or equal to 80, and the bridge is structurally deficient or functionally obsolete, it is eligible for HBP funds for rehabilitation only. The bridge must not have been replaced or rehabilitated within the last 10 years utilizing any federal funding source.

However, to be consistent with the program direction established in legislation and the Federal Highway Administration's interest in reducing structural deficiencies, the rehabilitation or replacement of structurally deficient bridge decks (those with National Bridge Inventory Item 58 – Deck ≤ 4) is eligible for HBP funding. Such bridge work is to be considered eligible regardless of the bridge's sufficiency rating. The 10-year rule will not prevent a current structurally deficient deck from being replaced or rehabilitated; however, once the deck has been replaced or rehabilitated, the 10-year rule will apply.

Review the bridge inspection file and address all the deficiencies and problems identified in the file.

Pre-plan preparation and post-plan preparation bridge inspections are warranted for major (deck replacement and significant modifications to main load carrying members) rehabilitation projects. The bridge designer or design consultant must be present for these inspections. Depending on available data and complexity of the work, either of the two inspections may be waived by the District Bridge Engineer.

5.5.2 List of Items to be Included

For PPG projects, specific bridge items will be evaluated and corrected. Items to be evaluated include crack sealing, joint repair, seismic retrofit, scour countermeasures and painting structural steel. PPG projects should correct bridge deficiencies contributing to accident clusters. PPG will include the upgrade of guide rail connections to the bridges, guide rail protection at bridge piers and bridge safety walks to prevent vaulting.

When the repair needs are extensive, the bridge portions of an PPG project may be upgraded using normal rehabilitation criteria. The following list of items includes a description of items included in rehabilitation strategies.

5.5.2.1 Geometry

Bridge width: Refer to DM-2, Chapter 1. This includes criteria for Bridges on Very Low Volume Roads.

Vertical clearance: Refer to DM-2, Section 2.20

Barrier: Based on the Test Level (TL) required (See DM-2, Chapter 12.10) follow Standard Drawings BD-601M, BD-615M, BD-617M or BD-618M for barrier rehabilitation.

Guide rail transition: Follow Standard Drawing BC-739M, BC-703M, BC-707M or BC-708M for Interstate Pavement Preservation projects with the detail on Sheet 4 of BC-799M for CIP barrier details located off of the structure.

5.5.2.2 Drainage

(a) Superstructure

Refer to PP3.2.3.6. Ensure that the existing scuppers and downspouting are repaired, cleaned or replaced and splash blocks are provided if none exist.

(b) Substructure

For MSE wall abutments, provide drainage according to Standard Drawing BC-799M using only 100-year life pipe with watertight joints within the wall backfill. For other abutment types, provide drainage as necessary.

(c) Off Structure Drainage

Provide an appropriate roadway inlet to eliminate shoulder washouts in accordance with Standard Drawing RC-50M.

5.5.2.3 Decks

The deck replacement decision shall be made based on whether the bridge structure geometry is substandard or is functionally obsolete, or if other major 3R/4R or PPG-type of work on the associated segment of the highway is to be undertaken. Otherwise, the following guidelines shall be used in the decision making process.

(a) Concrete decks without overlays

The following guidelines are provided for general purpose use only. For comparative values for bridge decks, see Figure 5.5.2.3-1.

Collect the following information prior to determining the type of rehabilitation needed for a bridge.

- (1) Extent of concrete spalls, in percentage of deck area.
- (2) Extent of delamination in percentage of deck area and/or traveled lanes. Use infrared thermography, chain dragging, or other approved methods.
- (3) Salt content in top 1 in. and in the last 1 in. of concrete immediately above the top mat of reinforcing bars.
- (4) Air content. The lower the air content, the more important other deck quality factors become.
- (5) Concrete strength. Use concrete coring and testing, and Windsor probe results, if permitted.
- (6) Type of deck steel (black vs. coated reinforcement bar) concrete cover over the top rebar mat.
- (7) Half-cell electric potential or other methods of determining active corrosion, as permitted by the Department.
- (8) Age of the deck.
- (9) Type and location of major cracking indicating superstructure flexibility problems.

Items 1, 2, 3, 4, 6 and 8 are required in all cases. Collection of these information items may be stopped at any point when it becomes obvious that the deck must be replaced. The remaining items will be required depending upon the

deck size, condition, roadway network and scope of the project. Any deviation must be approved jointly by the District Bridge Engineer and the Chief Bridge Engineer.

Provide deck survey and other results in the format shown in Figure 5.5.2.3-2.

Use Figure 5.5.2.3-3 "Bridge Deck Rehabilitation Guide". The table shown is based upon the assumption that the concrete deck has black steel. For concrete decks with epoxy or galvanized rebars, see item (b). The Chief Bridge Engineer will decide on any deviations for Federal oversight projects, and the District Bridge Engineer will decide on any deviations for PennDOT oversight projects.

Where latex modified concrete overlay is to be provided, the deck shall be scarified. Scarify the deck in 1/4 in. deep passes to avoid structure damage (cracking and spalling) of the remaining deck and to eliminate pulverization of the concrete around the reinforcement due to the high pressures needed for more than a 1/4 in. pass. If Hydrodemolition removal concept is permitted, the required depth may be removed in one pass. If Hydrodemolition equipment is utilized, the equipment shall be a computerized, self-propelled machine that utilizes a high pressure water jet stream to provide a rough and bondable surface while removing all unsound concrete, rust, and concrete particles from any exposed reinforcement during the initial pass. Initial experience with Hydrodemolition has indicated that this treatment should provide the expected service life.

Refer to Standard Drawing BC-783M for Type 2 and Type 3 repairs, latex concrete overlay transition, and expansion dam retrofit for the latex concrete overlay concept, and BC-788M for scupper retrofit details.

(b) Concrete Decks with Epoxy or Galvanized Reinforcement Bars, with Spalls

Provide Type 2 or Type 3 repairs. Surface deterioration due to loading, abrasion and other similar activities can be corrected by concrete repairs and latex modified concrete overlays. Bituminous overlays are not recommended since they entrap contaminated moisture beneath the overlay and accelerate deterioration.

(c) Concrete Decks with Bituminous Overlay and without Membrane Waterproofing

Replace such decks when they have a condition rating of three or less. For preservation or rehabilitation projects, cores shall be taken and assessed per item (a) to determine scope of work.

(d) Concrete Decks with Bituminous Overlay and Membrane Waterproofing

Patch deteriorated concrete and replace affected membrane and overlay until the repaired area is anticipated to be 50% or more of the deck area. Follow Figure 5.5.2.3-3 thereafter.

(e) Concrete Decks with Latex Modified Mortar or Concrete Overlay

Follow Figure 5.5.2.3-3. Consider latex modified mortar or concrete overlaid decks the same as regular concrete deck when using this table. If a new overlay is required, the existing overlay and deteriorated deck areas must be removed, and appropriate patching and overlay should be provided as per Figure 5.5.2.3-3. Pressurized water jet concrete removal may be cost effective in such instances.

(f) Open Grid Steel Decks

Open grid steel decks have a poor performance record. Fatigue cracking of welded grid members have been a continual maintenance problem, even on routes with low ADTT. For existing bridges where continuous maintenance welding is experienced, replace deck with concrete filled grid steel deck or fill the existing deck with concrete.

Mechanical connections between the deck and the supporting beams is preferred over welded connections for ease of construction and minimization of weld cracking. This should minimize continued maintenance costs. Also specify shims to eliminate residual construction load stresses induced by the placement of a load over the grid.

Install anti-skid studs when needed.

Open end steel grid decking shall not be used for new construction, or deck replacement when the anticipated total accumulated truck traffic will exceed 300,000 trucks (exceeding 3 tons) per lane during the expected life of the flooring. It has a poor performance record and promotes rapid deterioration of the supporting members. Such decks may be used for a temporary bridge or as a temporary deck.

(g) Concrete Filled Grid Steel Decks

The use of grid reinforced concrete bridge decks is permitted within the limitations imposed by Standard Drawing BD-604M.

Generally, these decks last for a long time (40 to 50 years). However, in older designs the filled concrete may cup out after ten years or so, depending upon the quality of the original work and whether or not the concrete was placed above the deck steel.

Remove deteriorated concrete and fill the cups with concrete if the full-depth concrete is deteriorating. Use of calcium nitrate may be considered as a rust inhibitor. Fill the cups with overlay material if only the top quarter or less of concrete is deteriorated, and overlay the deck with either bituminous or asphalt as a short-term (i.e., < 5 years) or latex modified concrete overlay as a mid-term (10 ± years) or other approved polymer resin concrete overlay as a long-term (15 to 20 years) solution.

Also with older designs, longitudinal creeping problems of concrete filled grid decks is well documented, particularly with decks having large (more than three) aspect (span length/width) ratios. Timely deck cut-off and releasing the deck for expansion will minimize chances for secondary stresses in other bridge members. Provision for closely spaced expansion joints would reduce deck creeping. However, it can increase expansion joint maintenance and the probability of subfloor and substructure deterioration.

This deck type may be used for deck replacement in special situations only, since it is very expensive and longitudinal creeping is a major concern.

(h) Timber Decks

Replace the deteriorated members with treated lumber. If deck deterioration is over 25%, replace the entire deck with treated timber or other material. Bituminous overlay may be provided to improve riding quality. If overlaid, use a leveling course prior to providing the surface course.

(i) Other Deck Types

Rehabilitation of other deck types shall be evaluated and rehabilitation strategies shall be developed based upon the characteristic, performance and condition of such decks. In some instances, special studies will be needed.

For determining construction quantities, anticipate further deck deterioration from the day of field investigation to the actual deck work and anticipate considerably more Type 2 and 3 repairs than field observations indicate.

Deck repairs and rigid deck overlay or overlay repairs are eligible work under PPG criteria.

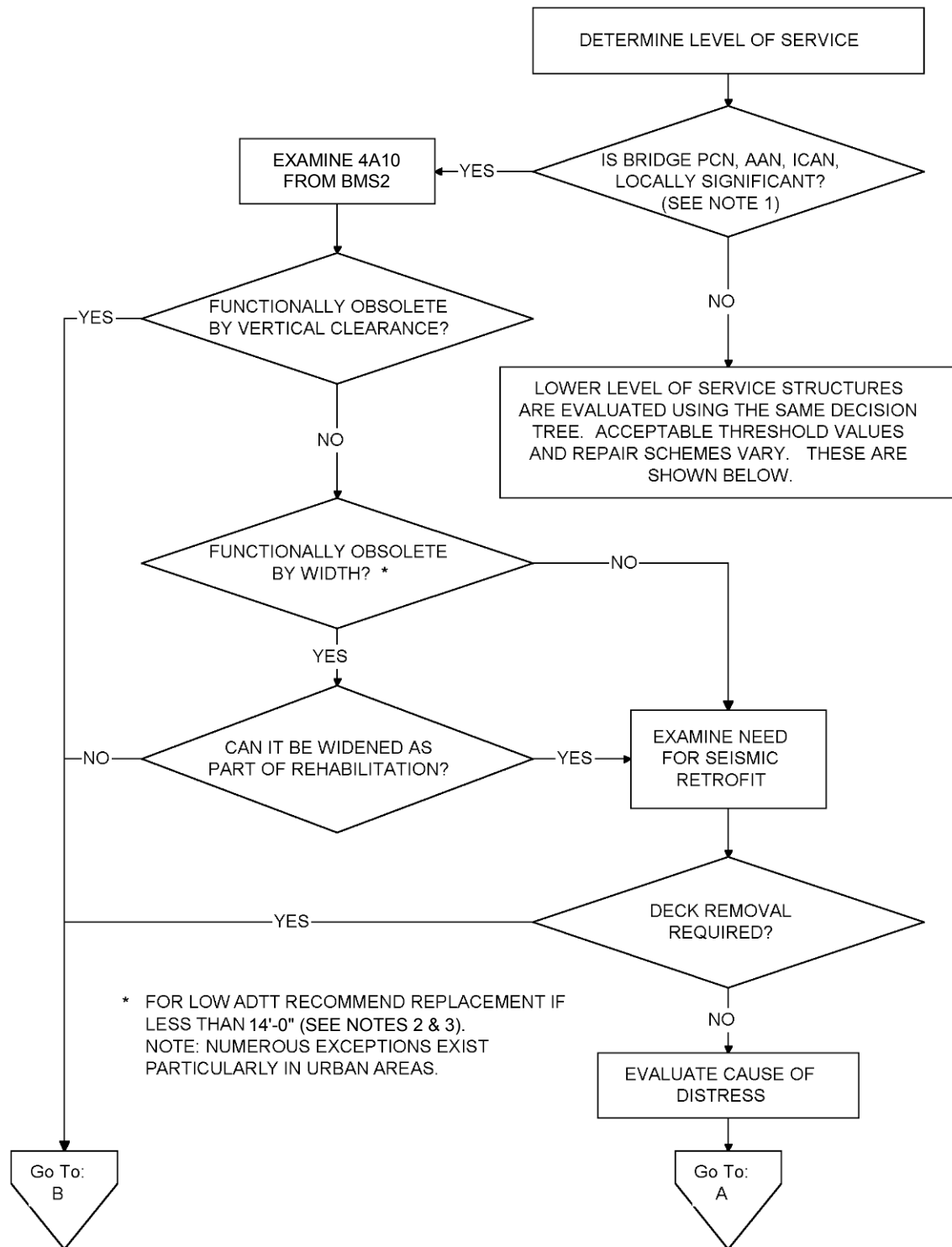


Figure 5.5.2.3-1 – Comparative Values for Bridge Deck

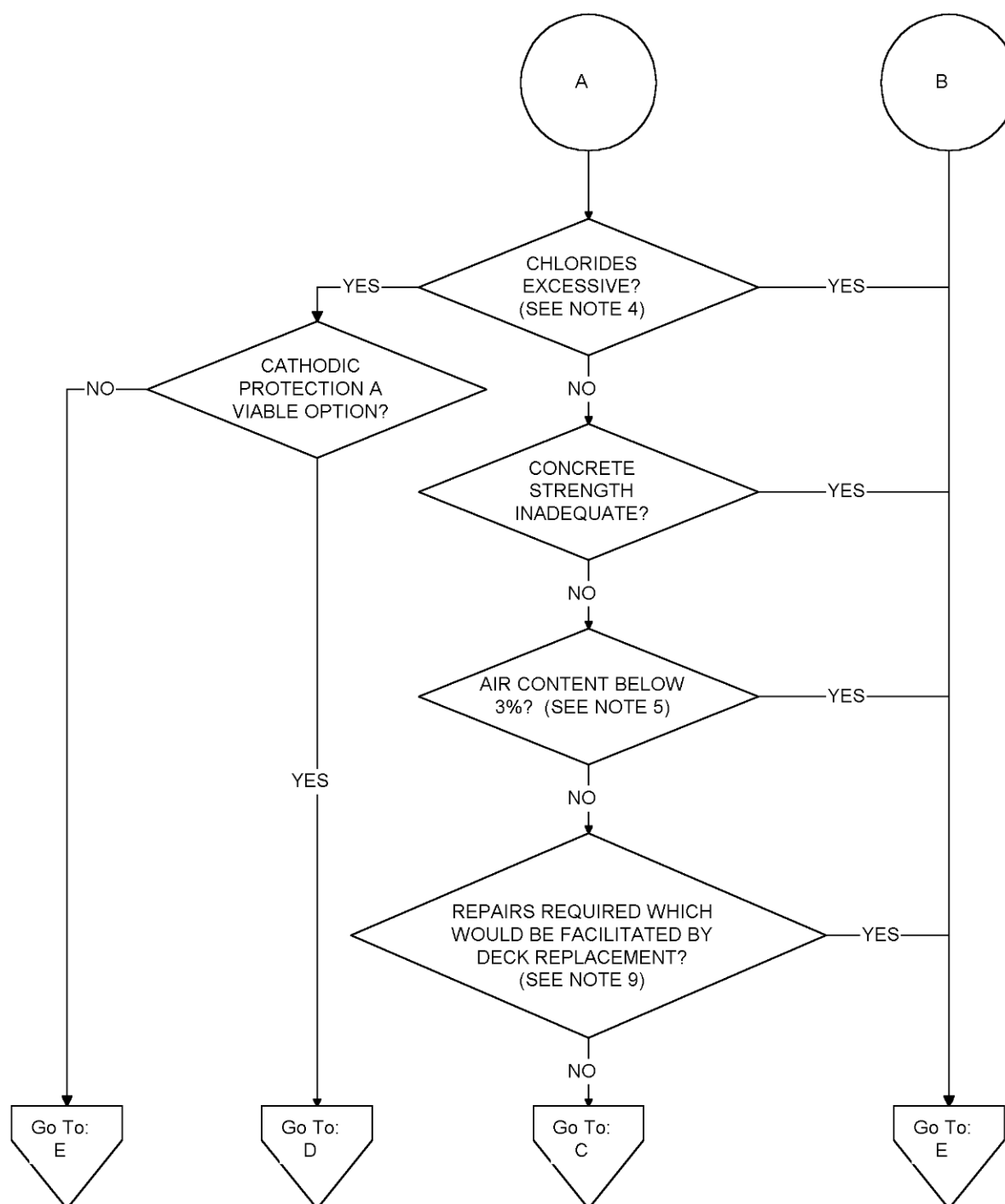


Figure 5.5.2.3-1 – Comparative Values for Bridge Deck (Continued)

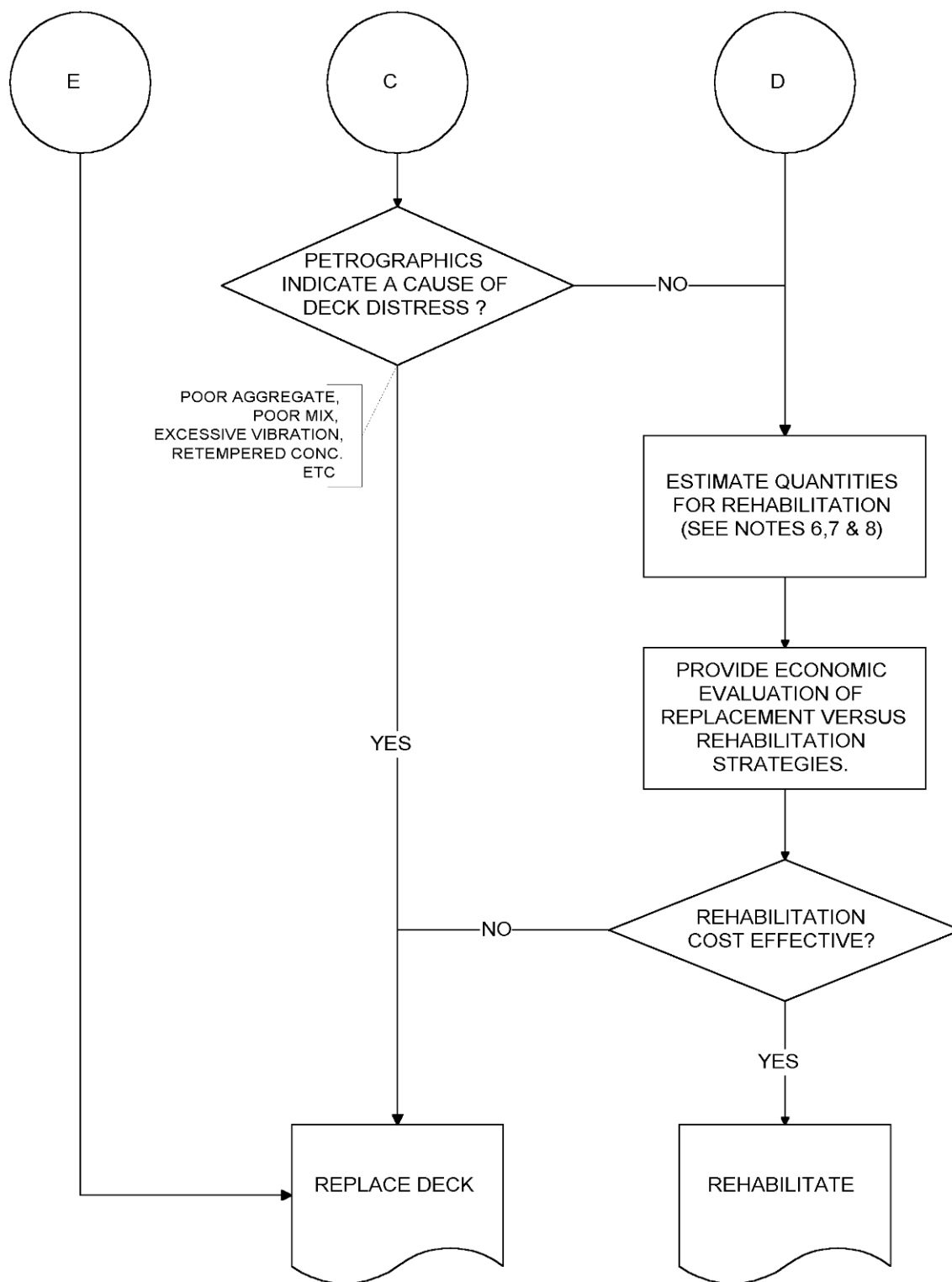
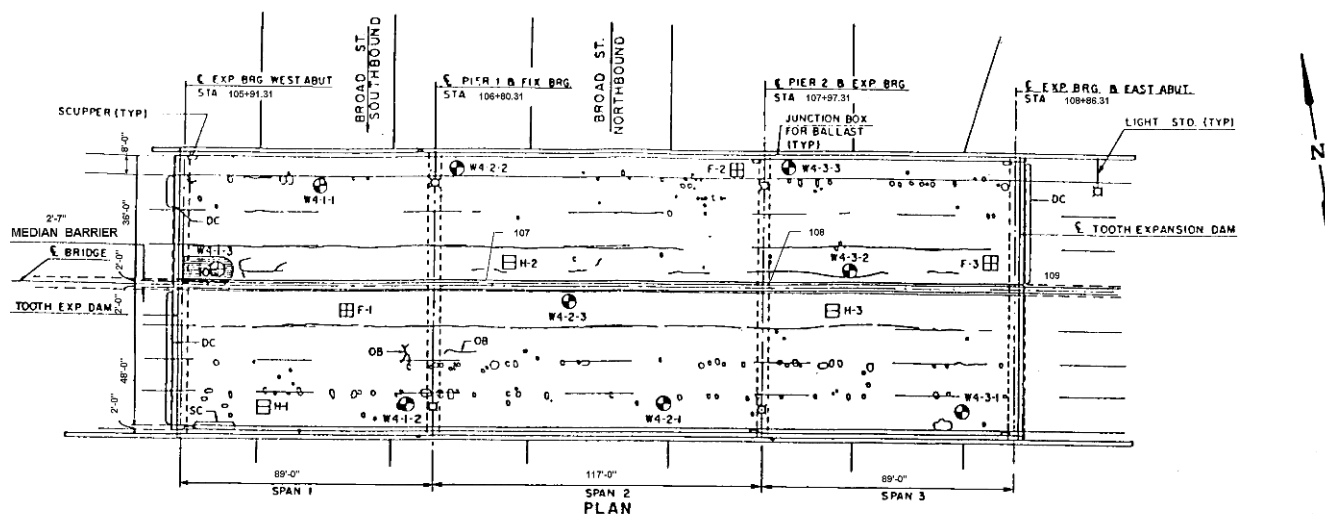


Figure 5.5.2.3-1 – Comparative Values for Bridge Deck (Continued)

NOTES:

- (1) Bridges which have significant impact on the local economy should be included in this group.
- (2) Reference, Ahlskog, O'Connor, "Level of Service Approach to Estimating National Bridge Needs", IBC-89-71.
- (3) Condition rating of 3 per BMS2.
- (4) Research studies have found chloride contamination accelerates corrosion when chloride levels are between 1.0 and 1.4 pcy. FHWA has not mandated threshold values for acceptable levels of chloride concentration. Contamination is often deemed as significant at the 1.5 to 2 pcy level. However, bridge decks have continued to perform well with 6 to 9 pcy. Higher levels of chloride contamination will be acceptable on lower level routes when programming rehabilitation/replacement strategies are considered. Each bridge will be evaluated independently.
- (5) Air content will be a factor when determining the salvage value of a bridge deck. On Business Plan Network (BPN) 1 and 2 routes, low air less than 3% will be justification for replacement. Also consider air characteristics. A surface area of air voids (specific surface) < 600 in²/in³ or a spacing factor of > 0.008 in. would also be justification for replacement. On lower level systems, low air content will be acceptable for certain structures, arches, etc. on lower level routes, low air content may be acceptable for other contents.
- (6) Estimated quantities for rehabilitation should reflect the anticipated quantities in the year of construction. Recommended rehabilitation schemes for high and medium ADTT routes include cathodic and latex modified concrete.
- (7) Estimated quantities for rehabilitation should reflect the anticipated quantities in the year of construction. Recommended rehabilitation schemes for low volume ADTT routes include bituminous overlays with membrane and latex modified concrete.
- (8) Bituminous overlays without membranes are viable options on all routes when the bridge deck has been programmed for replacement, and the overlay is used to provide a smooth riding surface until replacement can be accomplished.
- (9) Repairs would include bearing/pedestals, pier cap repairs, beam repairs, fatigue retrofits, etc.

Figure 5.5.2.3-1 – Comparative Values for Bridge Deck (Continued)

**LEGEND**

	CORE LOCATION
	CORE LOCATION TO CONFIRM THERMOGRAPHIC RESULTS
	FULL DEPTH REMOVAL
	PARTIAL DEPTH REMOVAL
	SEALED CRACK, UNLESS OTHERWISE NOTED
	OPEN CRACK
	OPEN CRACK WITH LEACHING
	ALLIGATOR CRACKS
	SLIPPAGE CRACKS
	"D" CRACKS
	POT HOLE
	RAVELING AND WEATHERING

GENERAL CONDITIONS

1. SHALLOW RUTTING CAN BE SEEN IN WHEEL PATHS.
2. AREAS ADJACENT TO SCUPPERS, CURB LINES AND EXPANSION DAMS HAVE BEEN SEALED.
3. LOW SEVERITY RAVELING AND WEATHERING, APPROXIMATELY 1" TO 2" IN WIDTH, IS PRESENT ALONG THE EXTERIOR CURB LINES.

TOP OF DECK DEFECTS

CURB TO CURB AREA 14,750 SF EASTBOUND RDWY. = 12,980 SF WESTBOUND RDWY. (SPANS 1 TO 3)					
SYM.	TYPE	EASTBOUND ROADWAY		WESTBOUND ROADWAY	
		AREA (Sq. Ft.)	% OF DECK SPANS 1-3	AREA (SQ. FT.)	% OF DECK SPANS 1-3
	DELAMINATION	216	1.5	100	0.8
	PATCH	0	0.0	0	0.0
	DEBONDED PATCH	0	0.0	165	1.3

EXPANSION DAM OPENINGS

LOCATION	EASTBOUND ROADWAY			WESTBOUND ROADWAY		
	THE OR. (IN.)	ACT. (IN.)	TEMP. (°F)	THE OR. (IN.)	ACT. (IN.)	TEMP. (°F)
WEST ABUT.	2 7/16	2 1/2	80	2 9/16	2 1/4	55
EAST ABUT.	2 5/16	2 3/8	80	2 11/16	2 3/8	55

CONCRETE CORE RESULTS							WINDSOR PROBE RESULTS	
LOCATION	CHLORIDE CONTENT				COMPRESSIVE STRENGTH		LOCATION	COMPRESSIVE STRENGTH
	DEPTH 1 (IN.)	PCY	DEPTH 2 (IN.)	PCY	H/D	KSI		
W4-1-1	0'-1"	1.7	2'-2 1/2"	0.55			1	8.200
W4-1-2	0'-1"	5.5	2'-2 1/2"	3.4	1.64	5.720	2	7.500
W4-1-3	0'-1"	1.1	2'-2 1/2"	0.51	1.74	5.970	3	7.800
W4-2-1	0'-1"	1.5	2'-2 1/2"	0.47			4	8.850
W4-2-2	0'-1"	0.94	2'-2 1/2"	0.27	1.80	6.250	5	7.600
W4-2-3	0'-1"	5.1	2'-2 1/2"	3.3	1.49	5.060	6	7.600
W4-3-1	0'-1"	1.5	2'-2 1/2"	0.78			7	8.025
W4-3-2	0'-1"	2.1	2'-2 1/2"	1.6	1.37	5.180	8	8.600
W4-3-3	0'-1"	2.6	2'-2 1/2"	2.4	1.54	5.410	9	9.500

Figure 5.5.2.3-2 – Format for Deck Survey and Other Results

Bridge Deck Rehabilitation Guide (1)

Deck Cond (2)	9 - 7						6 (6)						5						4						≤ 3					
% Air (3)	< 3%			≥ 3%			< 3%			≥ 3%			< 3%			≥ 3%			< 3%			≥ 3%			< 3%			≥ 3%		
Deck Age (4)	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971	Post 1976	1971-1976	Pre 1971
Rehabilitation/Replacement Strategies (5)																														
(LO)	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●												
(BP)																														
(BO)																														
(RO)								L	L																					
(RMO)								M	M			L	L																	
(R)							●			●																				
(RL)								H	H			M,H	M,H									M,H	M,H	M,H						
(RC)												H											M,H	M,H						
(RCMO)																							M,H	M,H						
(RCL)																							M,H	M,H						
(RPL)																			H				H	●	●	●	●	●	●	●

KEY

Where options appear in more than one row, an economic evaluation is required and/or additional test data to support a decision.

- - Denotes action for all occurrences (level of service) of a given condition rating.
- - With concurrence of the District Bridge Engineer.
- L - On low volume routes (ADTT < 100)
- M - On medium volume routes (100 # ADTT # 500)
- H - On high volume routes (BPN + Exceptions) (ADTT > 500)
- - Shaded area is "gray area" where other factors must be considered.

Follow procedures in Figure PP5.5.2.3-1

NOTES

- (1) The Bridge Deck Rehabilitation Guide as such is for generalized usage. Exceptions will exist. These may include: monolithic deck systems, arches, concrete tee-beams, etc.
- (2) Refers to deck condition rating established using BMS2 Item 1A01.
- (3) Refers to measured entrained air content.
- (4) Dates were established based on coated steel versus uncoated steel. Pre-1971 black steel was used in PA. 1971-1976 both epoxy-coated and galvanized, Post 1976 epoxy-coated steel was utilized.
- (5) Rehabilitation, maintenance and replacement options are listed per the following code:
 - LO = Linseed oil or other surface treatment (silane, etc.) at prescribed maintenance intervals (maintenance activity)
 - BP = Bituminous patching only (maintenance activity)
 - BO = Bituminous overlay only (maintenance activity)
 - RO = Repair Types II and III, plus bituminous overlay
 - RMO = Repair Types II and III membrane waterproofing, plus bituminous overlay (maintenance activity)
 - R = Repair Types II and III
 - RL = Repair Types II and III, plus latex concrete overlay
 - RC = Repair Types II and III, plus cathodic protection
 - RCMO = RC plus membrane waterproofing and bituminous overlay
 - RCL = RC plus latex modified concrete
 - RPL = Replace the deck
- (6) Intuitively the Benefit/Cost ratio on lower volume roads will not support high cost rehabilitation options.

Figure 5.5.2.3-3 – Bridge Deck Rehabilitation Guide

5.5.2.4 Expansion Joints

Eliminate expansion joints at substructure units whenever practical. The design life for bridge joints shall be compatible with deck life, which is currently considered to be between 40 and 50 years for decks with epoxy-coated bars or decks with similar deck protective systems and grid reinforced concrete bridge decks built in accordance with BD-604M. The deck joints incorporated in Standard Drawings BC-767M and BC-762M are expected to provide 40 to 50 years of life if properly fabricated and constructed. For maintenance projects with an expected service life up to 5 years, a two-part silicone joint system may be used. The expansion joint indicated in Standard Drawing BC-766M would provide reasonable life for low volume (ADT < 1000) roads. No other type of deck joint shall be incorporated without specific approval by the Chief Bridge Engineer.

Provide a trough or similar device under the existing tooth dams to protect the beams and substructure units from salt contamination and water.

Replace existing plate dams using either deck continuity or an appropriate joint specified in this section.

(a) Deck or Superstructure Replacement Projects

Minimize the number of joints by providing deck continuity over the existing joints and/or fixing the abutments, thereby eliminating backwalls if the criteria outlined in D11.6.1.7P is applicable. Refer to D14.5 for selection of the type of expansion joint.

The secondary effects of deck continuity, if done in conjunction with beam continuity, must be evaluated and corrective retrofits, if warranted, incorporated into the contract. Such effects may include effects on the type and size of bearings, superstructure to substructure connections, and structural capacity of substructure units including foundations.

Eliminate all pin-hanger joints and provide structural continuity. Effects of structural continuity on all superstructure and substructure elements, including foundation elements, shall be analyzed and appropriate structural modifications made.

For existing short-span prestressed concrete beam bridges (each span less than 80 ft.) having beam depth differences between the adjoining spans of 6 in. or less, consideration may be given to using only deck continuity. Full depth diaphragms shall be provided as per BD-664M and BD-665M at pier locations. In such instances No. 4 bars at 5 in. spacing longitudinal reinforcing steel in the top and bottom of the deck shall be provided as minimum steel for distribution to prevent transverse cracks.

For existing multi-span steel beam bridges, the designer shall consider providing continuity using flange and web connection plates at pier locations. In such situations, analysis of the girder as a continuous member shall be performed. The skew effect behavior of steel bridges and their relative flexibility when compared with prestressed concrete bridges must be evaluated and considered in the analysis.

At each pier, the beam end fixity must be the same at each of the beams. See Figure 5.5.2.4-1 for allowable configuration. Mixing of fixity (Expansion/fixed) is not allowed at these locations. Beam end restraints may require changes. Replacement of dual bearing lines with a single bearing should be considered particularly if replacement of the bearings is required.

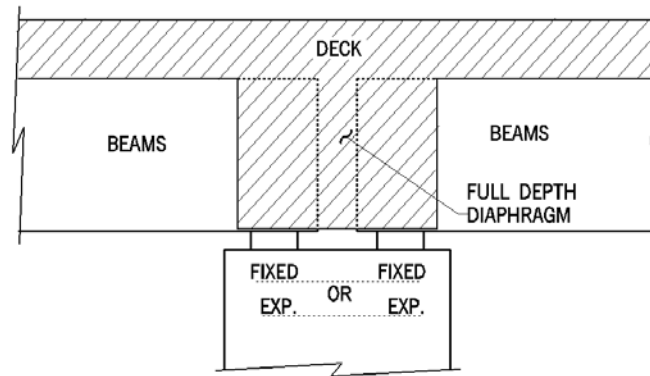


Figure 5.5.2.4-1 – Full Depth Diaphragms

(b) Deck Joint Replacement

Elastomeric deck joints that are found not to be repairable shall be eliminated by providing deck continuity or replaced with the joint types specified in Standard Drawings BC-767M, BC-762M or BC-766M.

In special situations shallow depth (1 1/2 in. or 2 in. deep) strip seal dams may be used with a specific approval from the Chief Bridge Engineer. Refer to Figure 5.5.2.4-2 for general details.

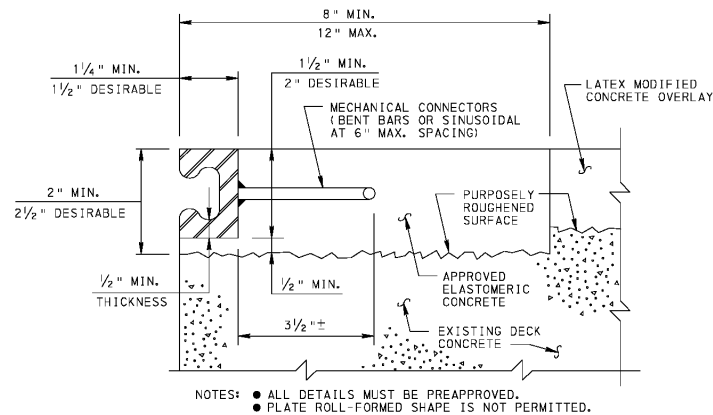


Figure 5.5.2.4-2 – Strip Seal Dams Detail

For bituminous overlays use Figure 5.5.2.4-3.

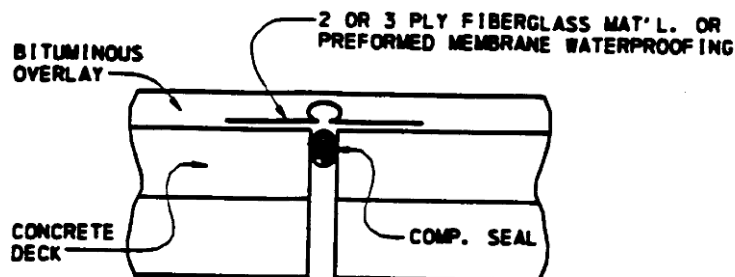


Figure 5.5.2.4-3 – Deck Joint for Bituminous Overlays

Replace damaged gland or troughs as necessary. Clean troughs. Non-performing expansion dams should be replaced with strip seal or other approved expansion dams.

(c) Elastomeric Expansion Dams

An analysis of the data for expansion joints using elastomeric expansion dams (received by May 31, 1994) revealed the following:

1. The first elastomeric expansion dams were installed in 1972.
2. Distress in these joints, in order of highest incidence were as follows:
 - a. Anchorage Failure
 - b. Concrete spalling at joint
 - c. Neoprene Failure
3. There was some indication that durability of the joints was related to truck volume. However, some joints on bridges having large truck volumes were outlasting those joints on bridges with small truck volumes.
4. Some dams had been replaced, others had been repaired, and many others required maintenance or replacement. A few were performing well.
5. A total of six suppliers furnished all the elastomeric expansion dams used in the State. They were:
 - a. Watson-Bowman Association
 - b. Fel-Pro
 - c. D. S. Brown Co.
 - d. General Tire
 - e. A. H. Harris
 - f. Royston

Appropriate rehabilitation strategies for these deck joints based on the type of rehabilitation required are as follows:

- Deck Replacement

When complete deck replacement is anticipated, consider elimination of deck joints first, where feasible. The decision to eliminate existing joints should be based on length of structure, type of bearings and substructure/foundation compatibility (see Section (a) of the article).

Where elimination of joints is not feasible, expansion devices, as shown on Standard Drawing BC-767M (Neoprene Strip Seal Dam, armored, for movements up to 4 in.), should be used. Tooth dams with 1/8 in. thick reinforced sheet neoprene trough shall be specified for movement over 4 in., as shown on BC-762M. The use of

a Preformed Neoprene Compression Seal Joint, unarmored, as shown on Standard Drawing BC-766M is discouraged, but may be used for structures having ADT less than 1000 and ADTT less than 100 with caution because its success in terms of providing a leakproof joint is highly dependent upon perfect construction.

- Joint Repair

There were many existing elastomeric dams that were performing well. However, some required maintenance such as anchor bolt replacement, full or partial seal replacement, hold-down plates or section replacement.

If a cost analysis shows that repair is cost effective, and the repair will restore the joint to water tightness, every effort should be made to schedule the maintenance.

Except for anchor bolts, replacement parts should be obtained from the appropriate suppliers to assure compatibility with the existing in-place dams. The use of steel plates to make temporary or makeshift repairs to damaged dams is not encouraged since the function of the dam to provide a water tight joint is not restored.

Generally, it has been found that the anchor bolt arrangements for the various dams were inadequate, particularly in areas of high stress (wheel paths). Consequently, bolts that have sheared off or pulled out should be replaced with new anchor bolts epoxied into pre-drilled holes. See Figure 5.5.2.4-4 for repair scheme.

Some elastomeric expansion dam failures originated from poor initial installation, particularly poorly consolidated concrete or uneven bedding in the block-outs. It is recommended that these areas be inspected during repairs to determine if other improvements should be made.

If temporary expansion dams are essential, asphaltic plug joints, elastomeric concrete with a two-part cold applied polymeric seal, or preformed silicone strip seals, as applicable, may be used as a stop gap measure.

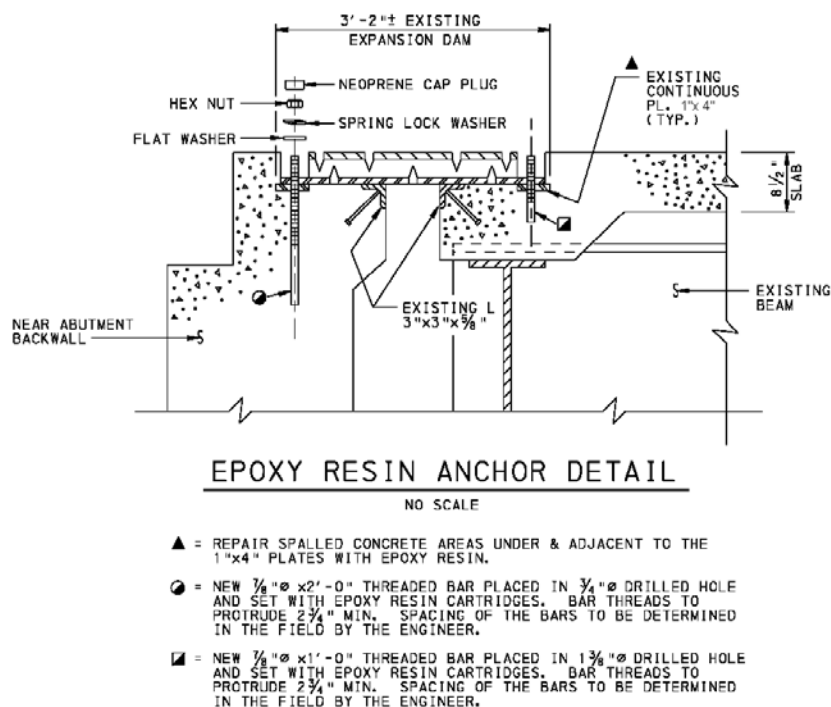


Figure 5.5.2.4-4 – Epoxy Resin Anchor Detail

- Joint Replacement

Elastomeric deck joints that cannot be repaired should be scheduled for replacement.

Several manufacturers have developed strip seal joints that are embedded in elastomeric concrete which can be used to bond the total joint to the concrete blockout. These proprietary systems include Delcrete, Wabocrete and Cevacrete, and are particularly well suited as replacement systems for existing elastomeric expansion dams because the required blockout depth is between 2 in. and 2 1/2 in., and the required blockout widths are comparable to the existing blockouts that accommodate elastomeric seals.

The above tradenames apply to elastomeric concretes furnished by the various manufacturers, i.e., D. S. Brown Co., Watson-Bowman Association and Epoxy Industries, Inc., respectively. The joints should be a combination of the elastomeric concretes, appropriate extrusions and compatible waterproof neoprene strip seals.

Strip seals are available for movements up to 5 in.. However, they should not be used for more than 4 in. movement classification due to our concern for load carrying capacity of wider openings.

Some of these systems use heat fusion bonding which will heat cure in about 2 hours, will bond to irregularities, such as spalls, in the blockout area, and are suitable for stage construction.

5.5.2.5 Bearings

(a) Rocker and Roller Bearings

For minor bridge rehabilitation work, unless replacement is warranted due to seismic requirements or functional obsolescence, readjust all rocker and roller bearings to restore their required function. Clean, paint and lubricate (roller bearings only) as warranted. For deck replacement or other major bridge rehabilitation projects, rocker bearings should be replaced and roller bearings should be considered for replacement unless seismic criteria is met.

(b) Other Metal Bearings

Restore the required function of these bearings, as warranted, by repairing or replacing worn-out parts. Special attention and analysis may be needed for bronze or similar friction parts.

(c) Pot Bearings

Ensure that the neoprene material is adequately contained in the pot and the gap between the top of the pot and the piston bearing plate is fairly uniform under dead load. Also, sufficient end distance should exist to the stainless steel plate (mirror plate) for expansion and contraction at extreme temperatures. If any deficiencies exist, corrective measures must be incorporated into the contract plans.

(d) Other Multi-Rotation Bearings

If adverse functional conditions exist, corrective measures must be incorporated into the contract plan.

(e) Neoprene Bearings

Problems such as major uneven deformation or walk-out shall be corrected.

(f) Bearings for Temporary Construction Condition

Appropriate bearing type and restraining connections shall be designed to endure construction and traffic loads. Expected life of the temporary bearing shall be five years.

5.5.2.6 Other Superstructure Elements and Fatigue

Establish material parameters based upon existing plans or previous testing. If data is not available, samples may be taken and tested to establish needed parameters. All construction details must be inspectable and maintainable.

(a) Redundancy

For non-redundant superstructure, ensure that all elements are structurally sound and will provide prescribed service life as specified in PP5.5.4. Where possible, an alternate load path should be provided if economically feasible. All pin-hanger connections shall be removed and replaced with continuity when replacing the deck as specified in PP5.5.2.4, item (a).

For redundant superstructures, the pin-hanger connections shall also be replaced when replacing the deck.

(b) Deteriorated Beam Ends and Painting

Deteriorated concrete beam ends shall be cleaned, repaired for structural integrity and protected from future deterioration either by deck continuity, encasing in concrete (diaphragm), or providing leakproof joints, and applying a breathable coating.

Deteriorated steel beam ends shall be cleaned, strengthened if needed, painted and protected from future deterioration by providing continuity or leakproof joints.

Where needed, spot and zone or total bridge painting shall be incorporated in the rehabilitation project to achieve the targeted life specified in PP5.5.4, unless a special painting contract is to follow very shortly after the rehabilitation project. Either the contract plans or special provisions must indicate whether or not the existing paint contains lead and other toxic materials such as cadmium, chromium, arsenic, etc., in order to alert the contractor. Paint coating coupons from different bridge members must be laboratory tested for lead content and other toxic materials such as cadmium, chromium, arsenic, etc. To determine cleaning and painting strategy, evaluate the thickness of the paint to be retained, adhesiveness and compatibility of the existing paint to the proposed paint system. For small span steel bridges with lead base paint, it may be cost effective to replace the superstructure.

To determine cost effectiveness, compare the remaining fatigue life, load carrying capacity, steel repair costs, cleaning and painting costs and other associated costs for the existing bridge, to the longer life and relatively minimal maintenance costs associated with a new superstructure. In borderline cases, permit a Contractor's alternate for a new superstructure.

(c) Cable Bridges

For cable-stayed and suspension bridges, cable condition must be thoroughly evaluated to ensure the targeted service life. Cables in the anchoring zone and splash zone are the most vulnerable. Deteriorated cables shall be replaced or reconstructed.

(d) External Post-Tensioning

External post-tensioning may be utilized to provide adequate load carrying capacity for short-term rehabilitation subject to special approval of the Chief Bridge Engineer. External post-tensioning is to be used as a last resort since the longevity of an unbonded system is questionable without periodic inspection. If external post-tensioning is employed, double corrosion protection to the prestressing tendons and end anchors shall be provided where possible.

(e) Jacking of Superstructure

Where superstructure jacking is required, at least one constructible option must be shown in the contract documents. All related analysis, including the effects of jacking on all connections, superstructure and (rarely) substructure elements must be evaluated. If strengthening is required, all details shall be shown in the contract documents. A Contractor's alternate may be permitted through a special provision or notes on contract drawings.

(f) Curb and Barrier

If the existing deck is to be replaced, construct standard curb and barrier.

Guide rail transition to the bridge barriers (Typical Concrete Barrier, PA HT, PA Type 10M and PA vertical wall barriers) shall be made as per Standard Drawings BC-739M, BC-707M, BC-708M and BC-703M. Guide rail transition to older style barriers (New Jersey shaped barrier) shall be developed using Appendix A of Design Manual, Part 2, Chapter 12, as the basis. Any exception shall be approved by the Director, Bureau of Project Delivery.

If the existing deck is to remain in place, it may be possible to modify the existing curb and barrier to meet the current standards. This decision should be made based on accident history, ADT and ADTT, approach geometry and sight distance, severity of the condition and the cost of improvements in conjunction with other factors evaluated during the scoping field view.

(g) Fatigue Evaluation and Retrofit

Determine remaining fatigue life of all critical details as specified in PP5.1, 5.2 and 5.3 and provide appropriate corrective measures.

For cover plate retrofit, refer to PP5.2.1.2. Retrofit all critical load and displacement-induced fatigue details. For displacement-induced fatigue retrofit details and load-induced retrofit details refer to PennDOT Research Project 83-21, "Deformation Induced Cracking in Steel-girder Bridges and Retrofit Guidelines", published in July 1987, and FHWA, March 1990 Publication, "Fatigue Cracking of Steel Structures", Volume II, Publication No. FHWA-RD-89-167.

(h) Utility Supports

Verify that all utility supports are structurally and functionally sound. They shall be either galvanized or coated with non-staining coating. If not, incorporate appropriate corrective measures.

(i) Structure-Mounted Signs

Ensure that all signs and sign connections are structurally sound. Specify repairs or modifications as necessary.

5.5.2.7 Substructure Elements and Scour

(a) Crack Sealing

Specify repair and/or rehabilitation of all deteriorated or damaged components. Special attention should be paid to cracked concrete pier caps, since the critical reinforcement for the pier caps may be vulnerable to corrosion if the concrete is crushed and exposed to contaminants from leaking joints. All such cracks shall be sealed with appropriate epoxy compounds.

If the cracking is caused by differential settlement, the situation shall be evaluated and corrected.

(b) Concrete Repair

Surface spalls of the concrete elements shall be cleaned to sound concrete and repaired with epoxy mortar. If deteriorated concrete extends beyond the primary reinforcement, the concrete shall be removed to at least 1 in. below the reinforcement and repaired with either concrete (if space permits) or lifts of epoxy mortar. An epoxy bonding compound shall be specified between the old and the new concrete and concrete lifts if needed. If significant deterioration exists, provide a temporary support to the superstructure and specify the needed repairs or replacement.

(c) Abutment Spalling

Spalling of the abutment stem under full-depth concrete diaphragm shall be repaired as depicted on Figure 5.5.2.7-1. Pavement migration should be corrected by providing a pavement relief joint.

(d) Hammerhead Pier Caps

External post-tensioning, as specified in PP5.5.2.6, item (d), may be specified for hammerhead pier caps to restore structural integrity. If a need for external post-tensioning is established, the system shall be designed to carry the entire load assuming the existing cap steel is ineffective. Adequate and uniform bearing between the concrete face and the post-tensioning system bearing plate shall be provided.

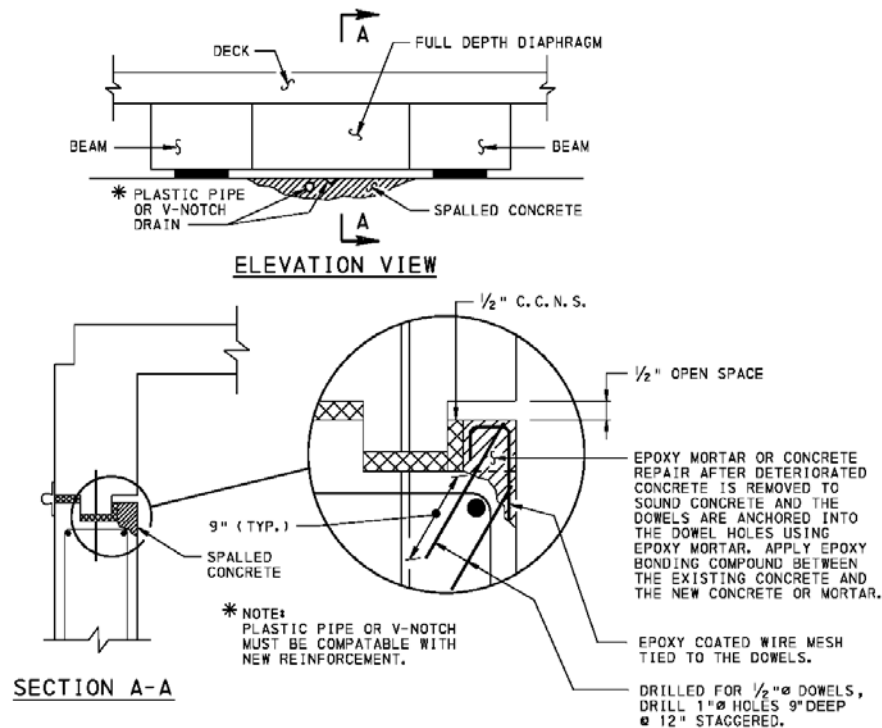


Figure 5.5.2.7-1 – Repair of Abutment

Other alternatives, such as casting wall or other support beneath the cantilever, may be feasible and may offer a long-term solution.

(e) Backwall Deterioration

Repair deteriorated backwalls. If the backwall is severely damaged due to pavement migration, it shall be replaced and a pavement relief joint installed.

(f) Structural Stability

If the existing substructure unit is determined to be marginal in overturning and/or sliding or overall slippage stability, the situation shall be corrected by appropriate measures. Generally, abutments on steep slopes are of concern. Such situations can be corrected by slope stabilization and the use of tie backs, replacement of the substructure unit, or by adding a span.

If settlement is a problem, the root cause(s) shall be determined and corrective measures shall be incorporated. Corrective measures may include underpinning, revising superstructure bearings if settlement has ceased, other appropriate corrective measures or replacement.

If settlement of a substructure unit founded on steel piles is evident, the integrity of the piles could be suspect. Settlement of point bearing piles in karst conditions may indicate sinkhole activity. Settlement of steel friction and/or bearing piles may indicate subsidence of substrata or corrosion activities of the piles. Settlement of friction or end bearing steel piles passing through fill materials may indicate corrosion, negative friction, overloading of piles, and settling substrata, or a combination thereof. Each situation should be evaluated on its own merit and corresponding corrective action shall be specified.

(g) Scour

Evaluate scour susceptibility and provide remedial measures as per HEC-18 and PP7.2.

(h) Cathodic Protection

For existing substructures with evident salt damage, the use of cathodic protection should be considered. Even though the technology of cathodic protection has advanced significantly over the past several years, it is still considered experimental, particularly for substructures. Cathodic protection of substructure units must be approved by the Structural Materials Section of the Bridge Design & Technology Division.

(i) Waterproofing Membrane for Substructure Units

If the juncture of the stem and footing of an abutment, retaining wall, or wingwall or juncture of a pier column and footing is exposed during preservation or rehabilitation of the structure, place waterproofing membranes at locations of below grade construction joints through which J-bars pass.

Place waterproofing membrane on the front face of the abutment, wingwall or retaining wall when the distance from the edge of pavement to front face of abutment, wingwall or retaining wall is less than or equal to 5 ft.. Place waterproofing membrane on the face of the pier column facing traffic when the distance from the edge of pavement to pier face is less than or equal to 15 ft..

5.5.2.8 Seismic Considerations

5.5.2.8.1 General

The 2010 AASHTO LRFD Bridge Design Specifications has revised the seismic methodology. Based on this methodology, Pennsylvania is classified as Seismic Zone 1. For Seismic Zone 1, the basic seismic rehabilitation requirements are:

- Minimum seat length
- Stable bearing types
- Adequate connection between the superstructure to substructure
- Proper reinforcement detailing of pier columns

It is the policy of the Department to require a seismic assessment on all rehabilitation projects. As part of this assessment, the Designer must indicate all deficient seismic items and provide preliminary details for any needed seismic retrofits. The Chief Bridge Engineer may permit a waiver of seismic retrofits on a project-by-project basis, if requested from the District Executive. The request must include a justification for waiving the retrofit and explain when the structure will be completely upgraded to comply with basic seismic requirements.

Please note that the Department has not developed standard retrofit details. Schematic examples indicated herein are acceptable. FHWA Research Reports FHWA-IP-87-6, FHWA-RD-83-007 FHWA-RD-94-052 and FHWA-HRT-06-032 Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges, also contain acceptable references for retrofit details.

To fulfill the basic seismic requirements, the following lists and sketches provide additional direction and guidance into common retrofit details to be used in Pennsylvania. These items are not to be considered exhaustive nor should they exclude sound engineering practice.

5.5.2.8.2 Common Retrofit Concepts

1. Replace “high” rocker and roller bearings. See Figures 5.5.2.8.2-1 and 5.5.2.8.2-2 for examples of these bearing types and an example of a typical replacement and retrofit of these bearings, respectively.
2. Extend bearing seats. Bearing seat lengths must meet the minimum support lengths as per the design specifications. This must be addressed on rehabilitation projects. Seat extensions, as per Figure 5.5.2.8.2-3, should be provided.
3. Provide cribbing for vulnerable bearings. While it is desirable to eliminate vulnerable bearings (i.e., rocker and roller bearings), the Department recognizes that this is not always possible or cost effective. Cribbing to assure support of the superstructure in the event of a bearing collapse is an acceptable alternate to complete bearing replacement. Cribbing is expected to be used as a temporary measure until an economical bearing replacement can be performed (i.e., during a deck or bridge replacement). See Figure 5.5.2.8.2-4. Note that the approval of the Chief Bridge Engineer is required if rocker and roller bearings are not replaced.
4. Add Shear Blocks and/or Pedestals - Structures which are deficient in areas such as seat length and bearing instability, or have inadequate superstructure to substructure connections may be retrofitted by addition of shear blocks and/or dowel bars, or by construction of concrete pedestals which will act as shear blocks and alleviate bearing instability. See Figures 5.5.2.8.2-9 and 5.5.2.8.2-10.
5. Department sponsored research has shown that piers and columns built according to pre-1992 AASHTO criteria should have acceptable seismic performance for all regions of Pennsylvania provided confinement reinforcement consists of a minimum of No. 4 bars at 12 in. and development and splice lengths meet current AASHTO requirements. If these conditions are met, no retrofit to the columns or piers is required. This does not mean that no damage will occur in the event of a ground acceleration of 0.15g, but the damage should not be life threatening and should be repairable (Memari et al. 2001). In cases where this minimum reinforcement is not present, see Figures 5.5.2.8.2-5, 5.5.2.8.2-6, 5.5.2.8.2-7 and 5.5.2.8.2-8 as acceptable means to retrofit this deficiency.

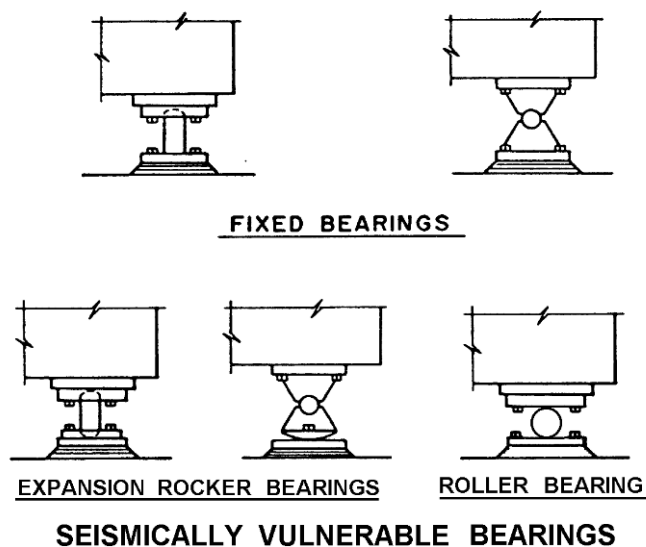


Figure 5.5.2.8.2-1 – Seismically Vulnerable Bearings

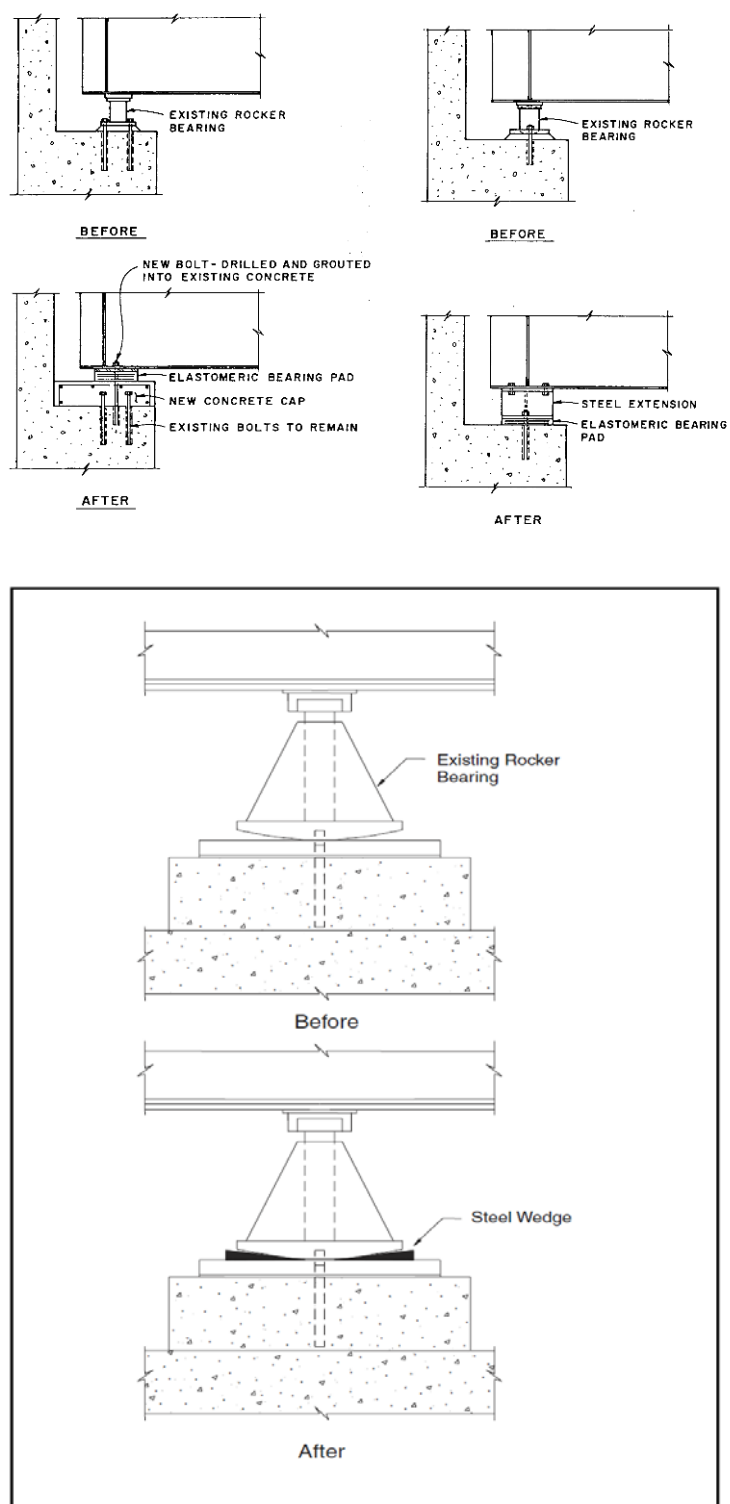


Figure 5.5.2.8.2-2 – Examples of Rocker Bearing Replacement and Retrofit

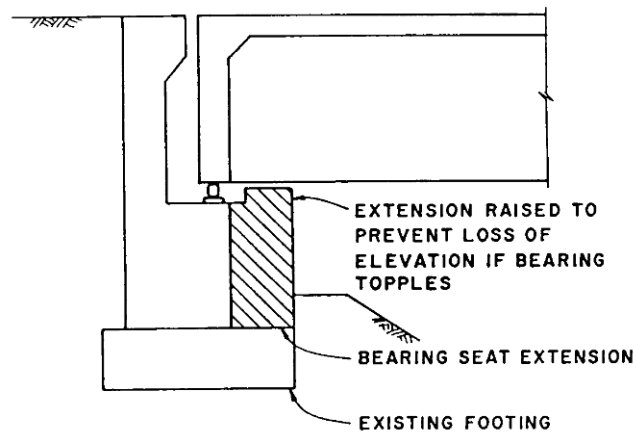


Figure 5.5.2.8.2-3 – Bearing Seat Extensions

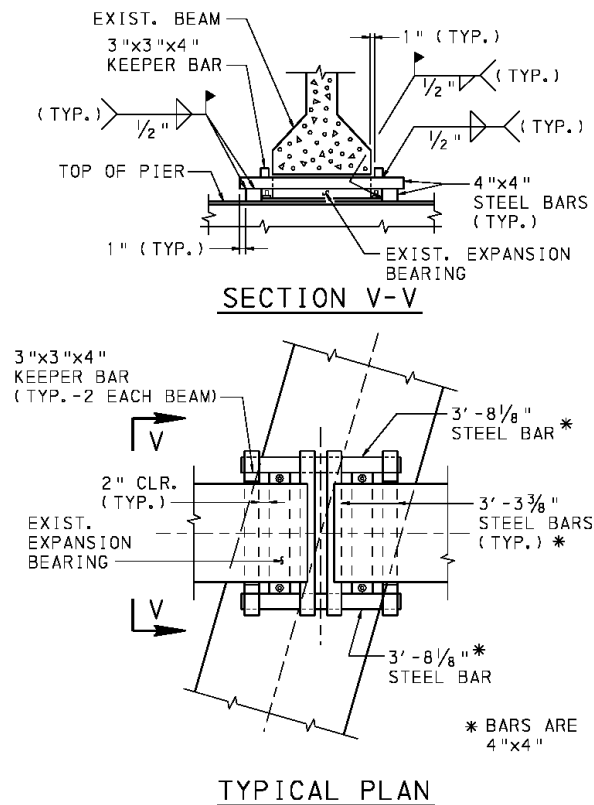


Figure 5.5.2.8.2-4 – Example of Cribbing for Bearings

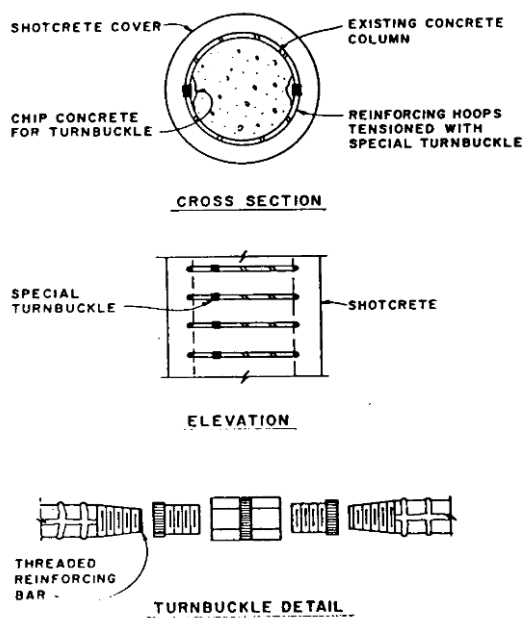


Figure 5.5.2.8.2-5 – Reinforced Concrete Column Retrofit-1

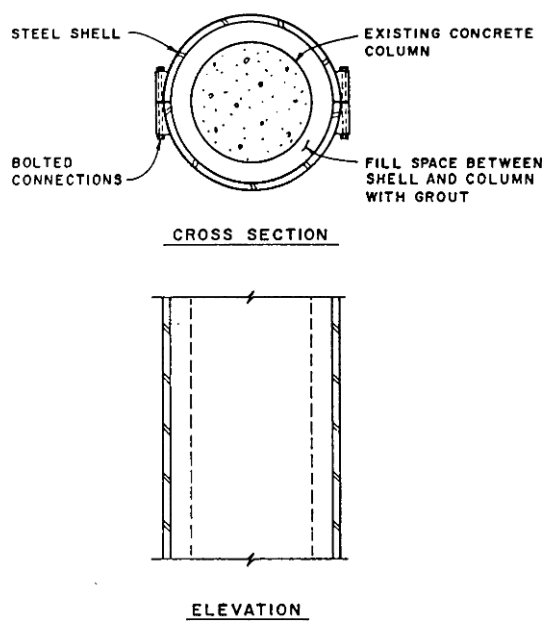


Figure 5.5.2.8.2-6 – Reinforced Concrete Column Retrofit-2

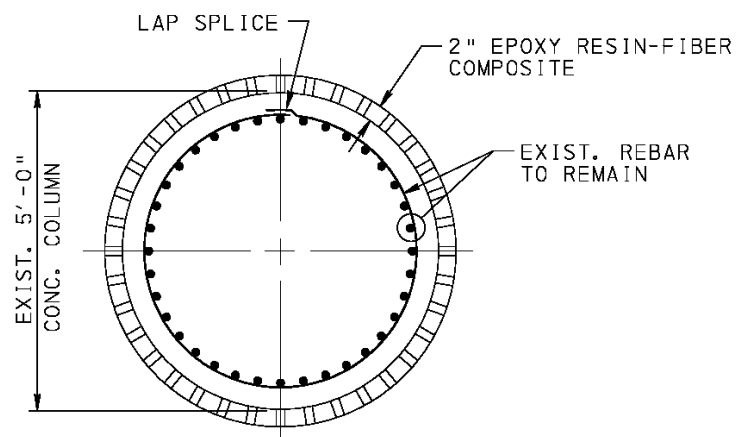


Figure 5.5.2.8.2-7 – Reinforced Concrete Column Retrofit-3

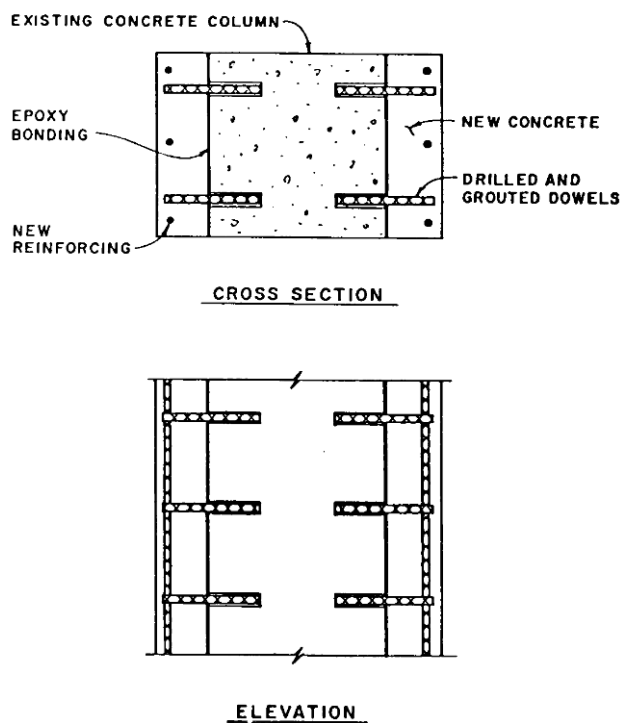


Figure 5.5.2.8.2-8 – Reinforced Concrete Column Retrofit-4

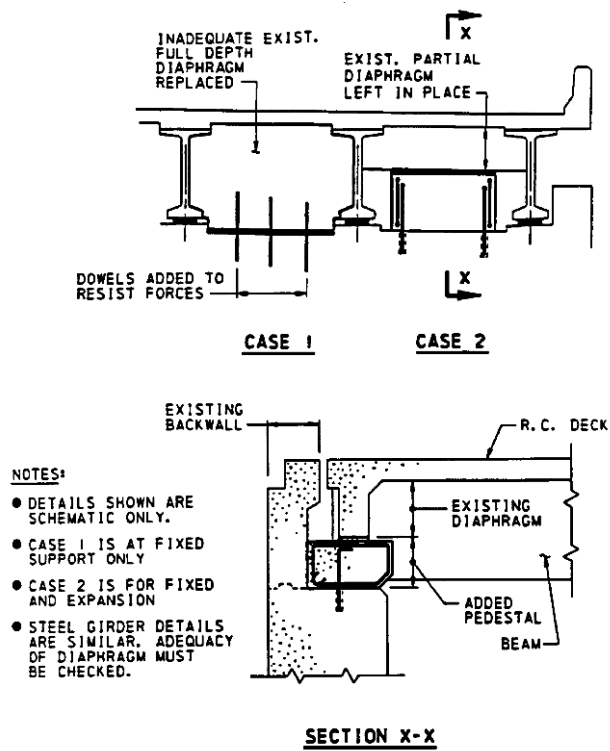


Figure 5.5.2.8.2-9 – Retrofit Using Shear Blocks or Additional Dowel Bars

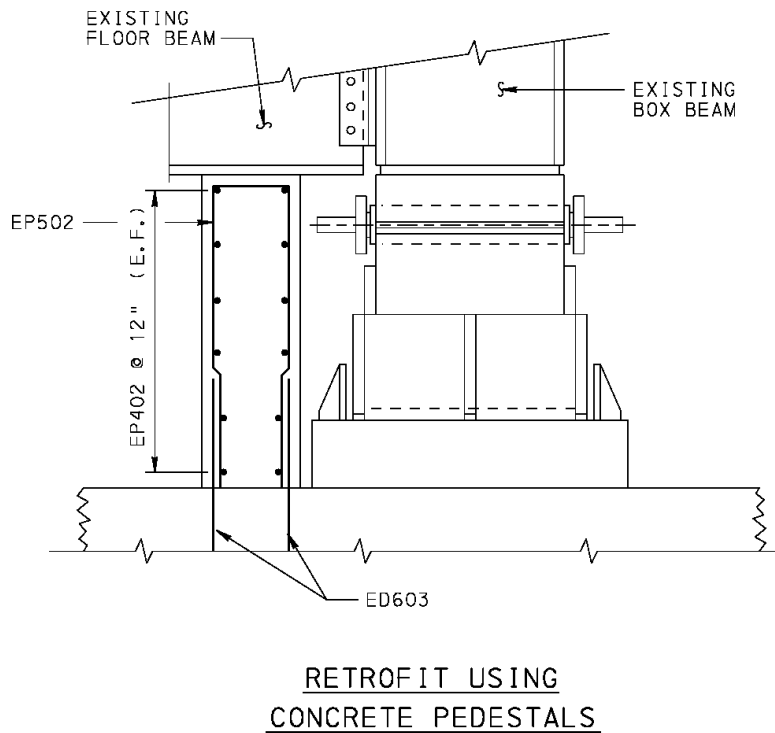


Figure 5.5.2.8.2-10 – Retrofit Using Concrete Pedestals

5.5.2.9 Bridge Approach Slab and Pavement Relief Joints

Replace or rehabilitate the bridge approach slab to help minimize the impact loading on the bridge.

Service the existing pavement relief joint as needed to ensure a smooth ride.

Provide a pavement relief joint for all concrete pavement types, if one has not been provided previously, except between two bridges which are less than 1,000 ft. apart, or when expansion length of the concrete pavement is less than 1,000 ft. apart.

5.5.2.10 Flared Safety Wings

Flared safety wings are eligible work under the PPG criteria.

5.5.3 Construction Related Items

5.5.3.1 Maintenance and Protection of Traffic

When an existing bridge is to be replaced or is to undergo major rehabilitation, the decision of whether to maintain traffic in the proximity of the existing bridge or to detour traffic must be made. This decision is based upon consideration of many conditions, including engineering feasibility, cost effectiveness, ADT/truck traffic, impact on local economy and emergency services, environmental impact and obtaining right-of-way.

The options for maintenance of traffic (including non-motorized modes as applicable) for bridge projects are to be evaluated and the decision is to be made during the Preliminary Design/Environmental study stage. Adequate public coordination must be performed in order to minimize adverse impact. If a temporary bridge is recommended, Program Management Committee (PMC) approval must be obtained prior to making a public commitment. A decision to use a temporary bridge at any time during the project is subject to PMC approval.

If, after completion of the Preliminary Design/Environmental Phase, a decision is made to use a maintenance of traffic option that was not reflected in the original environmental evaluation process, re-evaluation of the environmental impacts may be necessary.

Generally, maintenance and protection of traffic will be based on the following hierarchy of options. Refer to current Department policy for a complete discussion of these options.

1. Detour
2. Half Width Construction
3. New Bridge Adjoining the Existing Bridge (use existing bridge for maintenance of traffic)
4. Temporary Stream Crossing and Approaches (stream crossing using multiple pipes/fill material)
5. Temporary Bridge and Approaches (Temporary Bridge Structure)

Off-peak traffic hours construction schedule and/or incentive/disincentive clauses may be prescribed for bridges carrying extremely high traffic volume. Precast concrete deck elements with longitudinal post-tensioning or prefabricated steel decks may be utilized if warranted and approved by the Chief Bridge Engineer.

Traffic maintenance and related bridge construction items shall be depicted on plans. If temporary barriers are utilized, locate the temporary barrier on the structure plan. Based on the construction traffic patterns, select the barrier type from Standard Drawing BC-719M, and use the barrier to deck connections as shown on Standard Drawing BC-719M for the type selected.

5.5.3.2 Environmental Related Items

Requirements for paint removal, containment and disposal of contaminants shall be incorporated as per the current Department's policy.

Generally, it is the Department's policy not to construct sound abatement walls for existing highways. The need for bridge-mounted sound walls shall be determined in conjunction with the need for sound walls on the adjoining roadway.

The proposed construction should neither damage an existing wetland nor adversely affect the historical significance of the bridge itself or its surroundings, except as permitted through the environmental evaluation process.

5.5.3.3 Constructibility and Structural Stability

Constructibility shall include, but not be limited to:

- Material availability at reasonable cost
- Fabrication and erection requirements
- Site accessibility and material transportability
- Erection feasibility
- Construction risk
- Effect of the selected construction alternate on the project
- Construction sequencing of different operations
- Environmental impact of proposed construction method (including lead based paint issues)
- Impact on construction schedule

Each of the above items shall be evaluated to ensure constructibility and to minimize or eliminate "surprises" during construction.

For redecking projects, particularly when the new deck overhang may be larger than the existing overhang, structural stability of the fascia girder shall be evaluated using current design criteria.

For jacking requirements refer to PP5.5.2.6(e). If traffic is to be maintained on the temporarily jacked superstructure, the stability of the jacks or temporary bearing must be ensured by using needed restrainers and redundancy. Extra longitudinal and transverse forces due to traffic and other forces shall be shown on the contract plans for Contractor's use in proposing an alternate.

5.5.3.4 Temporary Bridges

If the need for a temporary bridge is established, each temporary road shall be designed to be compatible with the existing site conditions, volume and an acceptable operating speed for the temporary condition. Engineering judgment is to be used with these guidelines.

5.5.3.4.1 Definitions

For the purpose of these guidelines, the following definitions apply:

Temporary Stream Crossing – A temporary crossing of a stream with multiple pipes, pipe arches or similar conduits covered with fill material.

Temporary Bridge – A temporary crossing of a stream or other topographic feature consisting of a bridge superstructure with an appropriate substructure.

Temporary Road – A temporary roadway forming the approaches to a temporary stream crossing or temporary bridge.

5.5.3.4.2 Temporary Road, Traffic Control, etc.

For a temporary road, geometry and composition, traffic controls, permit requirements, environmental policies and other related requirements, refer to current Department guidelines.

5.5.3.4.3 Temporary Bridge Design Guidelines

Temporary bridges for public use will be designed using current Department bridge design methodology including consideration of geometric constraints (truck turning patterns, etc.) as per DM-2.

Temporary bridges can be specified to be constructed and removed by the Contractor or to be leased proprietary temporary structures if they meet the design requirements.

Normally, temporary bridges are a short-term installation, meant to be in use for a period ranging from a few months to two years. If a temporary bridge is being considered to be in place for more than two years, approval of the design by the Bureau of Project Delivery, BDTD must be obtained.

(a) Bridge Width

The minimum recommended clear width between curbs or bridges railing is:

- 14 ft. for single lane bridge
(A minimum width of 18 ft. may be required for farm equipment)
- 24 ft. for two-lane bridge

Bridge clear widths should not be less than the combined width of the temporary roadway and shoulders.

If proprietary temporary structures are specified (with special approval from PMC), clear bridge widths slightly less than those recommended above may be used.

(b) Bridge Railing

Bridge railing may consist of single face Typical Concrete Barrier, or other crash tested and approved railing system appropriate to the roadway (see DM-2, Chapter 12.10). Under most conditions, the Typical Concrete Barrier provides the highest and least costly level of protection. Therefore, the Typical Concrete Barriers are generally the preferred alternative. If the temporary bridge could cause hydraulics problems during flooding conditions, the use of an open metal railing (such as the PA Type 10M) is encouraged to minimize restrictions to water flow during high water episodes.

The connections between the bridge railing and any guide rail on the approaches is to be smooth and of adequate strength so that no "pockets" will be created if impacted by vehicles.

(c) Minimum Design Loads

The following are considered minimum design loads. If a temporary bridge must carry construction equipment, the appropriate loads must be considered in the design.

(1) PHL-93 at Strength IA limit state

- If the bridge is expected to carry heavy truck traffic and/or heavier loads, it must be designed for heavier loads (such as P-82 permit load at Strength II limit state).
- Use load factor of 1.35 for Contractor's heavy equipment.
- The Contractor shall be required to re-analyze the structure to ensure safety using these load factors if the bridge is to be used for construction equipment. Include a special provision in the PS&E requiring structural analysis to ensure prescribed factor of safety and bridge strength, if warranted.
- The temporary bridge must be posted in accordance with the current posting policy.

(2) No seismic loads.

(3) Other loads (wind, ice, etc.) as per LRFD or DM-4.

(4) Allowable live load deflection of $L/500$.(5) Debris loading shall be as per AASHTO Section 3.7.3.1. A Q_{10} flood will be used to calculate forces on the bridge.

(d) Substructure Design

The load factors for substructure design shall be multiplied by 0.90.

(e) Proprietary Temporary Bridge-Deck Surface

If proprietary temporary bridges are an option, specify that the bridge deck is to be coated to provide acceptable skid resistance (treated timber or steel plate decks do not provide acceptable skid resistance).

(f) Waterway Opening

A Q_{10} flood will normally be adequate. If warranted by site conditions and engineering analysis, the recurrence interval can be reduced (never lower than $Q_{2.33}$) or increased.

(g) Scour Protection

Scour protection will be provided in accordance with the scour analysis. Use Q_{25} flood to determine scour depth.

Include a special provision in the PS&E requiring the Contractor to close the bridge during high water. In the special provision, define high water as a specific water surface elevation. This water surface elevation will normally be based on a Q_{10} flood, but can be based on a larger flood (not to exceed Q_{20} if the designer feels the superstructure can safely tolerate the forces imposed by high water). Specify that the bridge may be reopened after inspection and adequate mitigation measures are taken (if warranted), and the bridge is determined to be structurally sound.

5.5.4 Targeted Service Life for Rehabilitated Bridges and Superstructures

(a) Estimated Deck Service Life

(1) Terminal decks (condition rating 3 or less) with minor patching and bituminous overlay.....2 to 5 years.

(2) Deck to remain in place with protective measures:.....20 years for deck

Membrane waterproofing and bituminous overlay. The life of the bituminous overlay may be 6 to 8 years. The membrane may need to be replaced each time the overlay is replaced.

(3) Latex modified concrete overlays, cathodic protection and rehabilitation of other deck types.....20 years

(4) New concrete deck with epoxy-coated reinforcement.....40 to 50 years

(b) Expansion Dams

Same as deck - periodic replacement of glands or trough should be expected.

(c) Beam end repairs and/or rehabilitation

Minimum: Same as deck

Desirable: 50 years

(d) Substructure repairs and rehabilitation

Minimum: Same as deck

Desirable: 50 years

(e) Repair and/or rehabilitation of other superstructure types and their elements

Minimum: Same as deck

Desirable: 50 years

(f) Bearings

Same as beams

- (g) New superstructure
 - Minimum: 50 years
 - Desirable: 100 years
- (h) Substructure rehabilitation
- (i) Same as superstructure
- (j) Retaining Walls
 - Minimum: 25 years
 - Desirable: 50 years
- (k) Culverts
 - Minimum: 15 years
 - Desirable: 50 years
- (l) New extension
 - Minimum: 50 years
 - Desirable: 100 years
- (m) Sign Structures
 - Minimum: 25 years
 - Desirable: 50 years
- (n) Ground-Mounted Sound Barriers
 - Minimum: 15 years
 - Desirable: 40 years
- (o) Structure-Mounted Sound Barriers
 - Same as deck
- (p) Temporary Bridges
 - 3 to 5 years

5.5.5 Load Carrying Capacity

5.5.5.1 Superstructures

Minimum load carrying capacity for all rehabilitated bridges shall be same as for a new design using LRFD/DM-4 method. Analysis should include 0.030 ksf for future wearing surface. A latex overlay is not considered structurally effective. Special approval of the Chief Bridge Engineer is required for any deviation.

Figure 5.5.5-1 shall be used for decision making for deck replacement projects.

If existing beams do not adequately rate for shear using the current LRFD shear criteria, the beams should be rated using the criteria used for the original design. A note on the rating table should indicate which criteria was used in determining the shear rating. Any design errors in the original design should be brought to the attention of the Chief Bridge Engineer.

All superstructure components must be checked for the remaining fatigue life. The remaining fatigue life must be at least the expected life of the type of rehabilitation being considered.

5.5.5.2 Substructures

Substructures should be analyzed for adequacy for the following conditions:

1. Superstructure Replacement
2. Change in Bearing Fixities
3. Evidence of Substructure Distress

The desirable load carrying capacity of substructures for rehabilitated bridges shall be HS-25. The minimum load carrying capacity is HS-20.

AASHTO criteria supplemented by DM-4 shall be used for the analysis.

Where the analysis of concrete substructures shows overstress using current AASHTO LRFD Bridge Design Specifications, the criteria used during the original design may be used. For shear analyses using working stress methods, do not exceed the stress limitations contained in the 1973 AASHTO Standard Specifications for Highway Bridges. Where no transverse reinforcement is provided (except footings), the current AASHTO Specifications for shear must be used.

Foundation bearing resistance should be determined based on available test boring data. The need for a detailed foundation investigation including the drilling of borings should be determined considering existing conditions, the size and complexity of the structure, and the extent of the proposed rehabilitation. If no boring data is available and no borings are planned for the project, as a minimum an assessment of the adequacy of the foundations to sustain the bridge in its rehabilitated condition should be made based on known substructure information and local geologic data.

Where end or point bearing piles are present, the pile resistance may be based on current design criteria. For friction piles, a static analysis should be performed using the available soil and pile data. In cases where the actual pile length or soil data is not known, an assessment should be made based on existing conditions and available data.

Deviations to this policy must be approved by the Chief Bridge Engineer.

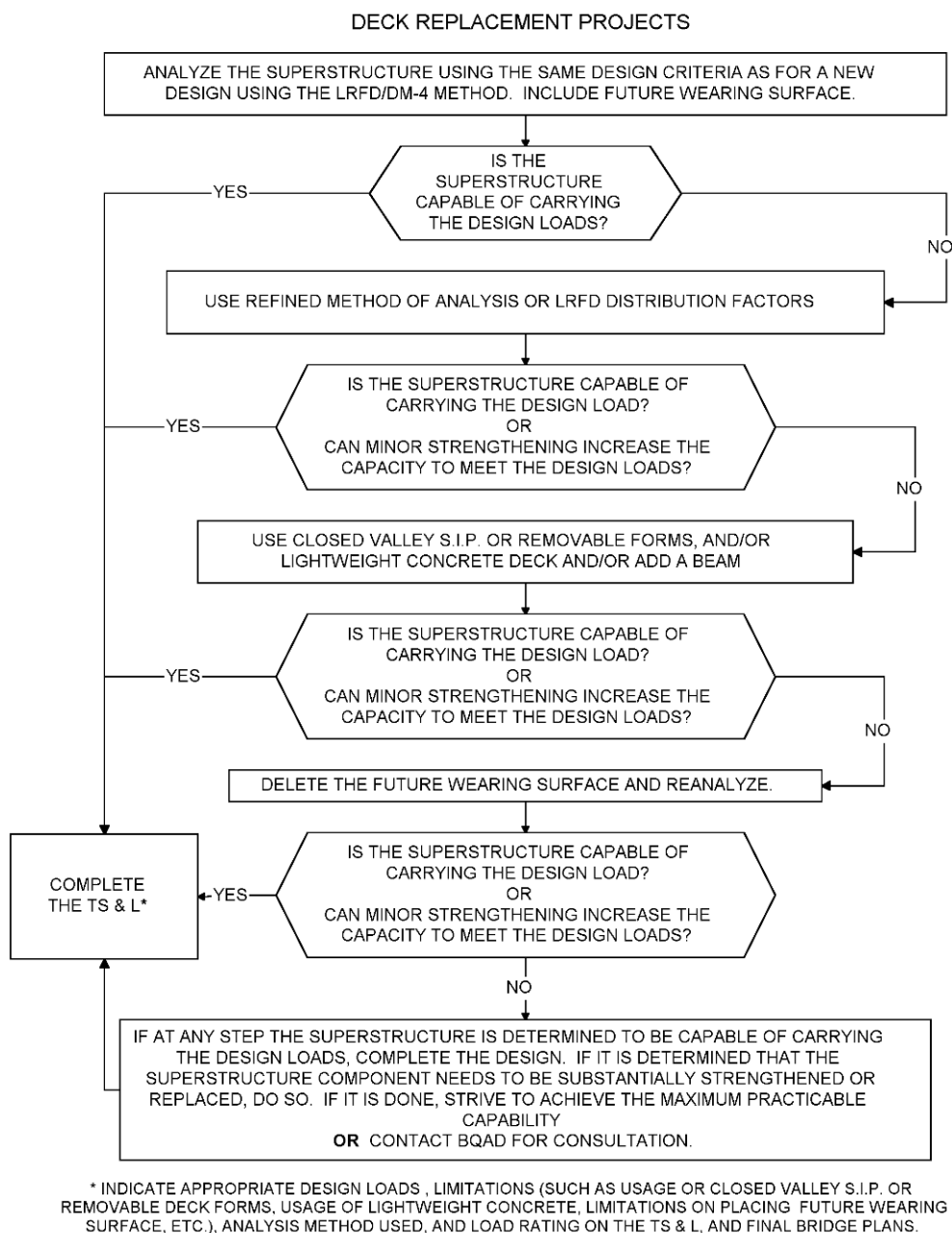


Figure 5.5.5-1 – Load Carrying Capacity for Deck Replacement Project

5.5.6 Plan Preparation and Presentation

For plan presentation follow PP1.9.6.2. The existing plans for rehabilitation projects may be obtained from the contract sales store. When developing bridge rehabilitation plans, all pertinent details should be shown on the contract drawings. In addition, the District should make the existing bridge plans available to the contractors during the bidding stage. Refer to PP1.3.2 for type of details needed for the existing bridge proposed to be rehabilitated. The pertinent notes from PP1.7 shall be shown on the proposed contract plans.

In plan preparation, actual field measurements should be considered more reliable than the drawings, and the shop drawings should be considered more reliable than existing bridge plans.

The contract plans shall be sufficiently detailed to provide an overall view of the bridge indicating the existing and proposed geometric dimensions, limitations and restrictions, extent and type of work to be performed, construction stages, material information, and all related information needed to rehabilitate the bridge. Pay limits, quantities and pay items

should be adequately defined to eliminate ambiguity or confusion. All work shall be accounted for by specific pay items and no work shall be hidden under "incidental" to other work item(s) unless for extremely minor work. The "incidental" work should be the exception rather than the rule.

Where applicable, reasons for critical limitations and restrictions should be explained to assist the Contractor and the field inspector in adjusting to the field conditions.

For submittal requirements and approval responsibilities, refer to PP1.9.

For rehabilitation projects, the TS&L plans shall have the normal TS&L plan details, plus a complete scope and extent of work, and the anticipated bridge rating after the proposed work is incorporated.

5.5.7 Retaining of Existing Beams by the Department

For bridge or superstructure replacement projects, FHWA has no objection to the Department retaining the existing beams if it so desires. Previous procedures of salvage value creditation has been discontinued. In fact, FHWA would encourage the Department to retain the existing beams whenever it would serve a useful purpose.

5.5.8 Bridge Rehabilitation Projects Requiring Bridge Painting or Superstructure Jacking

From a review of some project lettings, it is evident that, instead of cleaning and painting of the existing steel beams containing lead based paint, their replacement is sometimes cost effective. The cost of lead based paint removal and disposal is constantly increasing. To provide cost effectiveness, the following guidelines are provided for bridge rehabilitation projects involving bridge painting:

1. For projects requiring bridge painting, refer to PP5.5.2.6, item (b).
2. If the existing bridge paint contains lead or toxic materials, it must be indicated, either on the plans or in the special provision, to alert the contractor. If you are not sure of the lead or toxic material content in the paint system, take a few samples and send them to the Laboratory Testing Section of the Innovation and Support Services Division, for analysis. Based upon the test results, inform the prospective bidders, through bidding documents, whether or not lead is present in the paint.

For bridge rehabilitation projects where superstructure jacking is required, at least one constructible scheme must be shown in the contract documents. All related analyses, including the effects of the jacking on all connections and superstructure and (rarely) substructure elements, must be performed. Consider the following jacking design guidelines:

1. Bridge deck should be closed to traffic.
2. Do not include L + I loads to design jacking force requirements. Assume bridge is closed to traffic until jacking is done and bearings are completed.
3. If shims and blocks are used for temporary supports under traffic, their design must include L + I.

If strengthening is required, all details must be shown in the contract documents. A contractor may be allowed to submit alternate schemes through a special provision or notes on contract drawings.

5.6 BRIDGE PRESERVATION AND PREVENTIVE MAINTENANCE

The 2005 Safe, Accountable, Flexible, Efficient Transportation Equity Act (SAFETEA-LU) amended Title 23 United States Code Section 144 "Highway Bridge Program" (HBP) enabling systematic bridge preservation activities to be funded from the HBP. FHWA has determined that this HBP funding may also be used for systematic bridge preventative maintenance.

HBP funds may be used for these activities on any NBIS bridge. Other Federal funds may be used for these activities on NBIS and non-NBIS length bridges carrying roadways with functional classifications that are eligible for the specific fund category, but shall not be performed on bridges carrying local roads or rural minor collectors.

To obtain Federal funding the project must be competitively bid through ECMS.

5.6.1 Bridge Preservation

Eligible work items for bridge preservation are as follows:

1. Scour Countermeasures: Scour countermeasures including underpinning, riprap placement, streambed paving, etc. properly designed for predicted scour.
2. Expansion dams: Repair, replace or install new expansion dams to ensure leakproof joints. Where economically feasible, eliminate the deck joints. Repairs to deck drainage or downspouting may also be included. Replacement of seals is also permitted, provided other items, if any, relative to leakage are also addressed.
3. Beam end repairs and restoration: Restore steel, concrete or P/S concrete beam-ends to extend their service life.
4. Fatigue and Fracture Retrofits: Retrofits or repairs to fatigue-prone details of steel bridges.
5. Bridge bearings and supports: Restore or replace the existing bearings to make them functional and repair or rehabilitate substructure units to extend their service life. If bearings are replaced, they must meet seismic requirements. However, no seismic analysis is to be performed.
6. Spot/Zone painting: Spot/zone painting can be used as a stand-alone measure or with other steel repair items. Preservation of zinc-rich paint systems should be considered. Cleaning and waste disposal is included in this item. Spot/Zone painting to be completed in accordance with Pub 408, Section 1071.
7. Deck restoration and overlays: Concrete deck patching (Repair Types I, II, or III) and waterproofing overlays (i.e., latex concrete, bituminous with membrane) needed to extend deck life and improve rideability are eligible. Full deck replacements are not eligible. Bituminous deck patching alone is not eligible.
8. Painting: Full overcoats or complete repainting, with cleaning, waste disposal, and steel repairs in accordance with Pub 408, Section 1070.
9. Approach slabs: Repair the approach slab as necessary where the condition of the approach slab is affecting the performance of the bridge. Where practical and needed, repair or replace approach slabs, pavement relief joints, and other high spots adjacent to bridge to restore functionality and/or improve rideability.
10. Where practical, bridge preservation projects in close proximity should be grouped together to economize traffic control and other incidental costs. Bridges within limits of other highway work should be evaluated for opportunities for simultaneous bridge preservation work.
11. Other bridge preservation items not mentioned in the above categories may be included, but must be properly justified.
12. Safety items such as bridge parapet replacement are not eligible work items for preservation. However, safety improvements funded using other than bridge preservation funds may be included in such projects to take advantage of traffic control and other incidental project.

Once preservation activities are completed at a candidate bridge, this structure should not be revisited for rehabilitation or preservation for 10 years, except for routine maintenance activities.

5.6.2 Bridge Painting Guidelines

The corrosion of structural steel bridge members is an ongoing concern that must be addressed to prolong the life of the bridges in Pennsylvania. Not only does corrosion change the aesthetics of the bridge, it can seriously jeopardize the structural integrity of the entire structure. An efficient and economical method to provide corrosion protection to existing steel bridge members is painting.

The Bridge Painting Guidelines were developed to provide a baseline for programming painting projects to extend the life of steel bridges. The Bridge Painting Guidelines are primarily intended for use on bridges greater than 100 feet long. Packaging multiple bridges into one contract for structures less than 100 feet may be appropriate. For smaller bridges, the proportionally higher cost of environmental controls for cleaning may outweigh the benefits of painting. For larger bridges

(in excess of 500 feet) or complex bridges, paint preservation should be prioritized due to the high replacement cost of the bridge.

Bridge painting is weather sensitive. The air temperature must be warm and the humidity must be low. Therefore, work/letting need to be scheduled when there is low probability of inappropriate weather conditions. Typically, May through September is the ideal time to accomplish bridge painting. If a painting project occurs outside of this range, a controlled environment is required in accordance with Pub 408 section 1070.

Painting projects should be coordinated with roadway projects, especially on Business Plan Networks (BPN) 1 and 2. The necessary time for design and analysis of a containment system by a professional engineer registered in the state of Pennsylvania should be included in the project schedule between the notice to proceed and the physical start of work. Also, consider the necessary time required for the industrial hygienist/certified professional to develop/review the lead safety plan and other submittals. A typical lead removal painting project cost is in the range of \$14-18/SF (includes containment, lead removal, disposal, and repainting).

When repainting existing bridges over high ADT roadways where roadway restrictions must be minimized, use of a rapid deployment strategy should be considered. Rapid deployment is a viable option primarily designed for use on these highway overpasses where the structural steel is easily accessible from the roadway below using a mobile work platform. This mobile work unit includes a containment device, dust collector, and blast equipment. Rapid Deployment methodologies may be specified using Special Provisions. For field painting activities, use a three-coat system with an organic primer as per Pub 408, Section 1070. At this time, two-coat paint systems are not approved. Further research is necessary for single-coat systems.

5.6.2.1 Zinc Rich Paint Systems

For a properly shop-installed zinc rich paint system, the following painting activities and frequencies are general rules of thumb to establish painting guidelines to maintain and preserve the life of the steel bridges in Pennsylvania. Wide spread use of these zinc rich paint systems began in the 1980's. Environmental factors (e.g., under a leaking deck joint, within "splash zone", exposed to salt spray) will have a detrimental effect on the life of the paint system which will require an increased frequency of painting activities. Correct leaking deck joints and other bridge deficiencies affecting paint system performance prior to completing any new painting activities. Consideration must also be given to bridges that are on a program to be improved, rehabilitated or replaced. Bridges on a program must be evaluated to determine if a painting activity is still warranted. Due to the high cost of containment and mobilization, a cost/feasibility estimate must be completed to determine the most economic work scope for any given structure. (i.e., Use of spot/zone painting versus a full re-paint for any given structure or entire component replacement must be evaluated.) This work scope should include aesthetic considerations for the visible portions of the bridge, such as fascia beams. An overcoat painting activity is generally not an economically viable option for modern paint systems.

<u>Painting Activity</u>	<u>Frequency</u>
Spot/Zone Painting	10-18 years
Full Re-paint	30-40 years

Note: Maintaining the paint system on a bridge may require a series of spot/zone painting activities throughout the life of the paint system.

5.6.2.2 Lead Based and Non-Zinc Rich Paint Systems

The flow chart in Figure 5.6.2-1 is intended to assist in the selection of the painting activity for various paint condition ratings. For each activity, the candidate bridges must be prioritized and programmed. Funding level is an important consideration. For a description of the paint condition ratings, refer to Publication 100A, Bridge Management System 2 (BMS2) Coding Manual.

Some of the terminology used in Figure 5.6.2-1 is as follows:

6B36 – BMS2 Item for Paint Condition Rating.

Criticality Evaluation – This analysis/review is intended to determine if bridge painting is needed presently to preserve the bridge until the bridge is rehabilitated or replaced. This is primarily a structural safety issue.

Rehab / Replace – This option refers to the bridge being rehabilitated or replaced under a funding program, such as Billion Dollar Bridge, Betterment, I-4R, etc. This implies that the decision to paint or replace the steel will be included and implemented as part of that project.

Full Re-paint – This option involves total removal/cleaning and repainting of the entire bridge with a new zinc rich system.

Overcoat – This option involves the application of intermediate and/or top coats over existing paint with minimal removal of old paint. Compatibility patches are required at least one year in advance to determine the suitability of the proposed paint system.

Spot / Zone – This option is the re-painting of specific parts of the bridge, such as splash zones, near expansion dams, etc. Paint removal and cleaning will be thorough in those areas.

Do Nothing – No painting at this time.

Practicable – The term practicable means available and capable of being done after taking into consideration cost, existing technology, and logistics in light of overall project purposes. There may also be considerations (e.g., structural safety, historical preservation) that could over-ride pure economics.

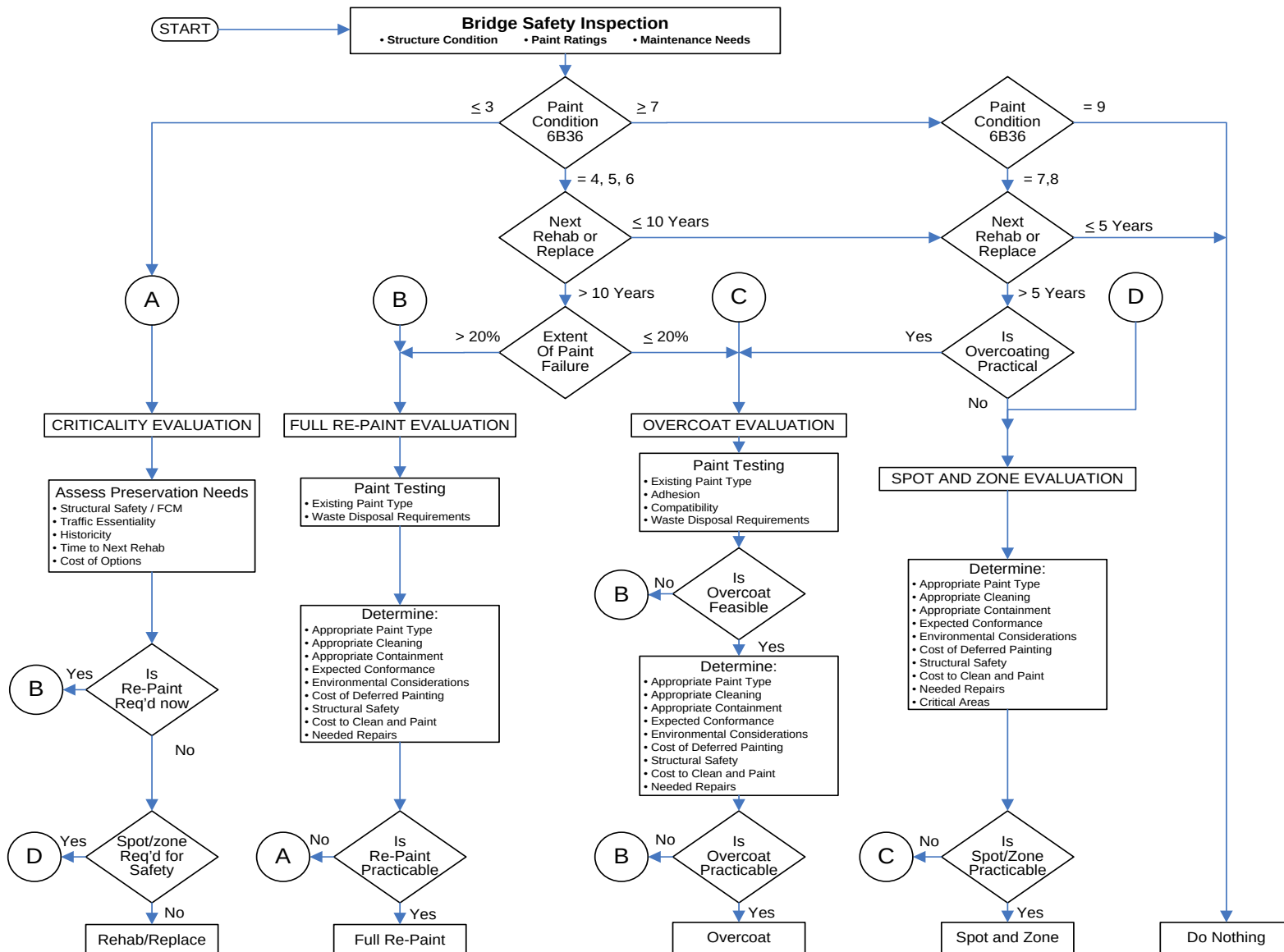


Figure 5.6.2-1 – Painting Activity Selection Flow Chart

5.6.3 Bridge Preventive Maintenance

5.6.3.1 Bridge Washing

The existing Title 23 Section 116 “Maintenance” prescribes that a preventive maintenance activity shall be eligible for Federal assistance if it is demonstrated that the activity is a cost-effective means of extending useful life. Therefore, preventive maintenance activities shall be demonstrated as cost-effective and shall be performed using a systematic process.

PennDOT has been performing bridge washing and cleaning for many years because it is a cost effective means to preserve our bridges and extend useful life. Bridge washing has been demonstrated as cost effective by New York State. Their bridges are exposed to similar environmental conditions as Pennsylvania’s including application of deicing salts. A report prepared by NYSDOT demonstrated a decline in deterioration rate from conducting the core preventive maintenance activities washing, joint repair and concrete sealing. A 40 percent decline in deterioration rate over a nine year period was denoted. The FHWA’s 2000 “Survey Results of Bridge Maintenance Practices in the Midwest” demonstrates that seventeen Midwestern States routinely perform washing because they believe it is cost-effective.

PennDOT’s methodology, systematic process, for selecting washing and cleaning candidates entails reviewing the maintenance screens in our Bridge Management System (BMS2) and reviewing inspection reports. Inspections are conducted on a maximum 24 month cycle and condition and maintenance needs updated. The inspection reports typically have a section that is dedicated to identifying specific detailed bridge washing needs. Eligible bridge washing and cleaning activities are identified in Table 5.6.3.1.1-1.

5.6.3.1.1 Bridge Washing Systematic Process

A systematic process shall be used to select bridges and their elements where washing and cleaning is performed as preventive maintenance.

Bridge washing and cleaning shall be performed on a specified cyclical basis or as-identified. Regardless if performed cyclically or as-identified, it shall be a consistent and continual program applied to the full inventory or a specified subset of the inventory. The effectiveness of this continual program shall be demonstrated by performance goals and measurements.

Cyclical activities will best occur following the cold weather deicing season. This is also an appropriate time as it improves surface cleanliness thereby decreasing safety inspection effort. As-identified (i.e. non-cyclical) activities shall occur based on findings discovered during inspection or maintenance activities or findings contained in inspection or maintenance reports.

Bridges programmed for complete replacement or superstructure replacement should be excluded from washing and cleaning unless necessary to prevent deterioration or debris/rust buildup from further affecting safe load capacity or functionality.

Table 5.6.3.1.1-1 – Eligible Washing and Cleaning Activities

Eligible Washing and Cleaning Activities				
Bridge Element	Element Location	Frequency	Benefit	Limitations
Deck surface	Shoulder or gutter line	Cyclical or as need identified	Prevent scupper/downspout blockages which contribute to system failure directing water & debris to deck joint or beam beneath scupper	Washing full deck width generally not cost effective especially on large widths given there is less permanent debris & contaminants in travel way
Deck joint	Top of deck joint	Cyclical or as need identified	Prevent premature joint failure from debris compaction &/or obstruction of movement	Strip seals, compression seals, modulars, sliding plates
Deck joint trough	Beneath deck joint	Cyclical or as need identified	Prevent premature trough failure from debris accumulation	Generally not needed if self cleaning performance demonstrated

Table 5.6.3.1.1-1 – Eligible Washing and Cleaning Activities (Continued)

Eligible Washing and Cleaning Activities				
Bridge Element	Element Location	Frequency	Benefit	Limitations
Superstructure elements beneath deck joint	Beneath deck joint	Cyclical or as need identified	Prevent corrosion or paint failure from visible & non-visible contaminants; necessary because deck joint integrity cannot be guaranteed	Generally steel members including beam ends, cross frames, diaphragms, floor beams, etc.
Seats & pier/abutment tops	Beneath deck joint	Cyclical or as need identified	Prevent corrosion of bearings, anchorages & reinforcement steel	Beneath deck joints unless need identified (low waterway crossings, etc.); may not be required for tops with intact coatings
Bearings	Beneath deck joint	Cyclical or as need identified	Prevent premature failure from corrosion; prevent obstruction of movement from pack rust or debris	Metal bearings or bearings with metal components (ex. pot bearings) beneath deck joints unless need identified (low waterway crossings, etc.)
Superstructure members	In vehicle spray zone	Cyclical	Prevent corrosion or paint failure from non-visible contaminants	Generally steel members; roadway beneath bridge is salted, has speed ≥ 30 mph & vertical clearance $\leq 25'$; include all steel beams & framing
Open grid deck	Open grid and elements beneath	Cyclical or as need identified	Prevent corrosion of grid decks, superstructure and substructure members from visible & non-visible debris & contaminants	If no visible contaminants cyclical frequency may be warranted when roadway is salted
Trusses, thru arches & other complex bridges	Members exposed to debris or salt spray; members prone to debris collection	Cyclical or as need identified	Prevent corrosion or paint failure from visible & non-visible contaminants	Generally steel members; many members are exposed to debris or salt spray including overhead & side members that form clearance envelope & members to the side & beneath the deck; many members are prone to debris collection including gusset connections, horizontal members, built-up open members, built up closed members with intermittent openings, etc.; most deck trusses & arches have less exposure

5.6.3.1.2 Bridge Washing Administration

The Scope of Work, Special Provisions, systematic selection process and list of bridges to be washed are to be submitted to BDTD for approval three weeks prior to advertisement.

5.7 PRESTRESSED CONCRETE BRIDGES

5.7.1 Repair of Prestressed Concrete Bridges

Department sponsored research on prestressed concrete girder repairs can be found in FHWA Report PA-2009-008-PIT 006, “Repair Methods for Prestressed Concrete Girders”. The department identified two repair methods that can be applied to the Department’s bridges from the numerous repairs investigated in the report. One of the selected repair methods will not restore beam capacity (repair of spalls and cracks) and one may restore beam capacity (damaged strands). The repair that will not restore beam capacity is the traditional concrete and mortar repair. The repair that may restore beam capacity is the non-prestressed/post tensioned carbon fiber reinforced polymer (CFRP) repair.

Repair and rehabilitation of damaged or deteriorated prestressed concrete beams, especially the repair of beams that were damaged by oversize vehicles is required on all rehabilitation projects.

The cause of the beam deterioration should be addressed to extend the design life of the repair. For example, leaky expansion joints must be repaired. A matter of concern is the observed deterioration of the bearing areas of some prestressed concrete box beams, generally found below leaky deck joints on structures usually more than 25 years old.

Spalling of the bearing areas is primarily caused by the infiltration of salt-laden runoff through leaky joints, with subsequent chloride saturation of the beam ends and resulting rusting of mild steel in the beam ends, and worse, rusting and debonding of prestressing strands.

Additionally, the beam seats at those deficient joints may be buried by 4 in. or more of salt-laden cinders and other flushed roadway debris. Nothing will stand up under such an environment. Structures thus affected should receive priority treatment under the Bridge Preventive Maintenance Program.

As an initial measure, the bearing seats and beam ends should be flushed clean and the joints should be repaired (by installing strip seal joints or eliminating the joint by providing a continuous deck as part of a rehabilitation project).

The affected structures should be monitored regularly as part of the inspection process, and joints, particularly on prestressed concrete box beam structures, should be kept watertight.

5.7.1.1 Repair of Prestressed Concrete Bridges (Spalls and Cracks)

Refer to Standard Drawing BC-783M for Reinforced Concrete Repair Prestressed Concrete Beam for repair details. A draft standard special provision for Prestressed Concrete Beam Repair has been developed and is under review.

If the repair area on the beam bottom flange at the bearing pad is greater than 10% of the bearing pad area, jack the superstructure off of the bearing pad during the construction of the repair. Jack and temporarily support the superstructure from the existing substructure, if possible, or construct a temporary support. Jack the entire end of the superstructure only the height required to insert a piece of galvanized sheet metal as a bond breaker for the new concrete repair.

5.7.1.2 Repair of Prestressed Concrete Bridges (Damaged Strands)

To be developed in conjunction with draft standard drawing “CFRP Strengthening Prestressed Concrete Beam” and draft standard special provision. The draft standard drawing and special provision are currently under review.

5.7.2 Procedure for Rating Existing Prestressed Concrete Bridges

The following steps shall be used in rating prestressed concrete bridges:

1. Analyze the bridge using PennDOT's LRFD Prestressed Girder Design and Rating computer program using the rating option.
2. Review analysis, does the bridge meet the operating rating levels?
 - Yes, summarize and submit findings
 - No, continue
3. Analyze the bridge using PennDOT's LRFD Prestressed Girder Design and Rating computer program using the rating option with this modification.
 - f'_c value increased by 20%

4. Review analysis, does the bridge meet the operating rating levels?
 - Yes, summarize and submit findings
 - No, is the P-82 rating vehicle the only vehicle which is not meeting the operating rating level [Currently, the program analyzes the bridge with the P-82 vehicle in all lanes. As of this writing, the program is being revised to apply the P-82 in the design lane that produces the largest load effects with the design truck in the other design lanes, effectively eliminating the need for Steps 4 and 5.]?
 - Yes, continue
 - No, go to Step 7
5. Analyze the bridge using PennDOT's LRFD Prestressed Girder Design and Rating computer program using the rating option with these modifications.
 - f'_c value increased by 20%
 - P-82 live load which is comprised of the P-82 in the design lane that produces the largest load effects with the design truck in the other design lanes
6. Review analysis, does the bridge meet the operating rating levels?
 - Yes, summarize and submit findings
 - No, continue
7. Perform rating of the bridge using the original design method.
8. Review analysis, does the bridge meet the operating rating levels?
 - Yes, summarize and submit findings
 - No, contact PennDOT immediately so that posting procedure can be started

REFERENCES

Keating, P. B., and Fisher, J. W., Review of Fatigue Tests and Design Criteria on Welded Details, Fritz Engineering Laboratory, Report No. 488-1 85, Table 4

Memari, A.M., Harris, H.G., Hamid, A.A. and Scanlon, A., "Seismic Column Reinforcement Study," Final Report No. FHW-PA 2001-015-96-29 (Vol. 1 & Vol. 2) submitted to Penn DOT, July 2001, Vol. 1, 310 p., Vol. 2, 352 p

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART A: POLICIES AND PROCEDURES

CHAPTER 6 – PROCEDURES FOR GEOTECHNICAL EXPLORATIONS

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6.1 GENERAL

Procedures followed for geotechnical explorations shall meet the requirements of this section, plus the following publications:

- (a) Design Manual, Part 3, Chapter 5, Sections 0 through 3
- (b) Design Manual, Part 4, Design Specifications, D10
- (c) Design Manual, Part 4, Policies and Procedures, PP1.9.4

Refer to Publication 293, Geotechnical Engineering Manual, for the technical aspects of the exploration, including potential problem areas and geologic hazards.

6.2 RECONNAISSANCE

For other than single-structure replacements, reconnaissance for soils and geological engineering exploration shall be performed as a part of the Preliminary Engineering Studies (see Design Manual, Part 1, Chapter 7, Section 7.1). Major features and pertinent data may be presented on topographic maps of the site areas. For single-structure replacements and similar small projects, the reconnaissance may be conducted during the hydraulic study.

The reconnaissance shall generally consist of the following five phases:

- (a) Search of published and unpublished information
- (b) Visual site inspection
- (c) Development of subsurface exploration program
- (d) Subsurface exploration
- (e) Preliminary Geotechnical Engineering Report

The following guidelines are provided to implement a core boring contract effectively:

1. Provide certified inspectors for drill rigs in accordance with Section 3.1 of Publication 222. Do not compromise the duties nor the responsibilities of the inspector.
2. Department personnel or a consultant working for the Department may provide the required drilling inspectors.
3. The contract document is extremely flexible. Hence, modify contract specifications on an as needed basis to secure information. Do not ask for information that does not contribute to determining the bearing capacity. Even though geotechnical studies are required during the preliminary design stage, ask for only minimum information at this stage to help to evaluate alternates. Seek detailed information for determining the bearing capacity or other soils parameters during the final design stage.
4. The standard contract requires filling each drilled hole with cement concrete or mortar. Use this clause judiciously. If use of other economical materials serve the intent, modify the contract requirement. In some instances (particularly NX coring in wilderness), filling holes may not be warranted. At the time of this writing, it is the Department's understanding that DEP does not require these holes to be filled.
5. The following items provide guidance on the geotechnical information required for various foundation types:
 - (a) If a spread footing is contemplated to be placed on bedrock (preliminary study would give some idea), detailed properties of the overburden material may not be of much importance, hence, only blow count, soil composition, gradation, water level, etc., would suffice for the overburden. However, all needed properties, including seam size, rock and seam quality, cementation, voids, solution cavities, reason for low RQD, or core recovery, etc., must be secured for the bedrock strata in accordance with the D10.

- (b) If a spread footing is contemplated to be placed on soils, all required data of the soil strata below the anticipated bottom of the footing up to prescribed depth should be secured to help in determining bearing capacity, settlement, and other pertinent values (e.g., corrosion resistivity for MSE walls). For bridges over streams, scour depth may control the bottom of the footing location and type of foundation.
- (c) For footings to be supported on point bearing piles (on bedrock), follow the same guidelines as for the spread footing on rock.
- (d) For footings on end bearing piles, secure detailed data of the pile end (last 10 ft. to 15 ft. of piles) supporting strata and the strata immediately below the supporting strata. Ask for minimum data (blow count, driveability constraints, corrosion resistivity, etc.) for the overburden.
- (e) For footings on friction piles, get all required detailed information for most strata.
- (f) For footings on caissons, secure all data for rock socket substrata and areas below it, bearing strata for bell type of caissons, and needed data for overburden for lateral stability similar to end bearing piles.
- (g) For footings on micropiles, secure all data for rock socket substrata and areas below it and needed data for overburden for lateral stability similar to end bearing piles.

The above guidelines are shown for uniformity. However, areas where special conditions, such as solution cavities, mines, sinkhole activities, etc., are expected will require more core borings than the normal requirement. Collection of sufficient data should not be compromised. Similarly, for tie-back walls, additional holes are needed in the tie bearing area and in the wall area. These guidelines will reduce construction problems, delays and work orders that have been experienced previously.

6.2.1 Search of Published and Unpublished Information

Review of available published and unpublished information shall include evaluation of the following:

- (a) Preliminary plans of the proposed construction
- (b) Surface features on topographic maps
- (c) Geologic maps and other sources of geologic information
- (d) Soil survey maps
- (e) Aerial photographs
- (f) Mine maps and subsidence information
- (g) Previous geotechnical explorations in the vicinity of the project, including any soil studies done during the environmental evaluation for the project
- (h) Logs of existing borings and water wells
- (i) Records and photographs regarding the construction and behavior of nearby structures relative to planned structures

In addition to review of the above information sources, consult with local residents, consultants, contractors and Department personnel who may be knowledgeable about local surface and subsurface conditions.

6.2.2 Visual Site Inspection

The information obtained in PP6.2.1 shall be verified and supplemented with a site inspection, preferably with the District Geotechnical Engineer. Observations of the following shall be included:

- (a) Proposed locations of structures

- (b) Existing structures (type and condition)
- (c) Surface soils
- (d) Topography and vegetation
- (e) Drainage features
- (f) Rock outcrops, excavations, visible indications of subsurface conditions
- (g) Existing problem areas, such as slope movements, subsidence, mine shafts, or sinkholes
- (h) Utility locations

6.2.3 Development of Subsurface Exploration Program

The Project Geotechnical Manager/Engineer (PGM/PGE) shall meet with the District Geotechnical Engineer and District Bridge Engineer to identify and summarize potential problem areas and to outline the planned subsurface exploration work. The District Geotechnical Engineer shall notify the BOPD of the meeting if it is anticipated that the expertise of the Geotechnical Section will be needed. A written submission is not required for the meeting.

A letter outlining the work shall be prepared by the PGM/PGE and submitted to the District for review and concurrence of the District Geotechnical Engineer before the work is begun.

6.2.4 Subsurface Exploration

For major projects, subsurface exploration shall be conducted at each proposed site to identify potential design or construction problems and to allow comparison of the suitability of the various sites. For single site replacements, foundation records from the existing structure will usually suffice, and subsurface exploration will not be required provided that no evidence is available indicating structural distress from the foundation. For structures where records are not available, a literature and file search should provide sufficient information.

A limited drilling and sampling (or test pit excavation) program shall be included, with at least one boring or test pit for each structure.

Laboratory testing of samples will rarely be required in the reconnaissance exploration. The scope of the investigation shall be sufficient to anticipate the type and depth of foundations, especially as related to structure type and span. For this work, subsurface exploration shall be considered a professional service.

The District Engineer may approve modification to, or elimination of, the subsurface exploration requirement if sufficient information is available from other evaluations obtained in the reconnaissance program to determine an economical bridge foundation.

6.2.5 Preliminary Geotechnical Engineering Report

Sufficient information about each site shall be presented to the Department in a Preliminary Geotechnical Engineering Report, developed in accordance with Publication 293, to permit a comparison of approximate construction costs for each site under consideration. Four copies of the report shall be provided to the District Geotechnical and Bridge Engineers by attachment to the Type, Size and Location (TS&L) submission. As a minimum, the report sections(s) shall include the following:

- (a) Proposed construction for which the exploration is made
- (b) Data obtained during the search of published and unpublished information, including information from available geomorphological studies conducted during environmental studies.
- (c) Subsurface features and structures observed during the visual site inspection and, if appropriate, air photo interpretation
- (d) Scope of the subsurface exploration, if conducted (see PP6.2.4)
- (e) Identification of geological formations in the area

- (f) Conditions encountered in the subsurface exploration
- (g) Evaluation of the effect of the condition encountered or anticipated on the proposed construction
- (h) Relative advantages and limitations of each site under consideration
- (i) Topographic map showing the proposed sites and locations of important site features
- (j) Indication of further exploration requirements (including boring layout)

Space shall be provided in the report for endorsement by the District Geotechnical Engineer.

6.3 FOUNDATION EXPLORATION - FINAL DESIGN

Following acceptance of the appropriate sections of the Preliminary Geotechnical Engineering Report, the foundation exploration shall be initiated as part of the final design in accordance with PP6.3.1 through PP6.3.5 and Publication 293. Publication 293 also provides guidance regarding selection of the number and depth of borings, drilling techniques, sampling methods, in situ testing, and laboratory testing.

The following steps should be taken as outlined in Publication 293 and herein:

- (a) Foundation exploration plan
- (b) Foundation exploration meeting
- (c) Preparation of proposal/contract
- (d) Administration of the contract
- (e) Report - Subsurface Conditions at Structure Locations (Note that this report is part of the Preliminary Geotechnical Engineering Report)

6.3.1 Foundation Exploration Plan

After reviewing all available information from the appropriate sections of the Preliminary Geotechnical Engineering Report and receiving approval for initiating the final design, the PGM/PGE shall prepare and submit to the District four copies of the preliminary plan. This submission shall be made at least two weeks before the final exploration plan meeting. For each site, the following shall be included:

- (a) The number of test borings at each structure unit.* For limestone and other similar problem areas, the following coverage shall be incorporated as a minimum:
 - (1) Test borings must "enclose" foundation.
 - (2) Additional borings shall be provided, as necessary, between substructure units to obtain an overall picture of the area at the bridge site.
 - (3) Selected holes shall extend to a sufficient depth to locate mines, caverns, or other underground openings which could affect foundation performance.
- (b) A discussion of possible adverse and positive subsurface conditions*
- (c) Profile sheet noting existing and design grades
- (d) Recommended methods of subsurface exploration
- (e) Plan sheet noting suggested test locations*

- (f) Recommended laboratory testing program
- (g) Copy of pertinent foundation information from Preliminary Geotechnical Engineering Report

*These items and the appropriate sections of the Preliminary Geotechnical Report shall be included with the TS&L Submission (see PP1.9.3)

6.3.2 Foundation Exploration Meeting

The PGM/PGE shall meet with the District Geotechnical Engineer and the District Bridge Engineer to present and discuss the Foundation Exploration Plan. The District Geotechnical Engineer and the District Bridge Engineer may request assistance from the BDTD of the BOPD, if the foundation investigation will be complex and involve special testing.

A field view shall be held if deemed necessary by the District Geotechnical Engineer. The PGM/PGE shall finalize the exploration plan on the basis of the conclusions of the meeting and submit the final exploration plan to the District as a part of the TS&L submission. The final exploration plan is the draft Subsurface Boring, Sampling and Testing Contract.

The Foundation Exploration Plan may be modified if unanticipated subsurface conditions are encountered during the drilling operation.

6.3.3 Entry onto Railroad Right-of-Way

6.3.3.1 Temporary Railroad Right-of-Entry Permit/Agreement

Entry onto Railroad right-of-way (property) may be required for construction activities as well as inspection, maintenance, survey, soil borings, engineering studies, etc.. Therefore, in order to gain entry onto Railroad property, the Department's contractor and/or his subcontractors or the Department's consultant and/or their subconsultants may be required to enter into a temporary Railroad Right-of-Entry (ROE) permit/agreement with the Railroad.

Because Railroad ROE permits/agreements have been determined to be a contract and they are not in compliance with the Commonwealth Attorneys Act, the Department cannot sign such documents. Only the Department's contractor or consultant who is performing the required work activities on the Railroad's property can sign a ROE permit/agreement. Refer to the latest edition of Publication 371, Chapter 7, Section 7.03 "Entry onto Railroad Property".

6.3.3.2 Department Scope of Work Details and Method of Payment

The following activities may be required by the Department's District Grade Crossing Engineer/Administrator (DGCE/A) or District Project Manager (DPM) regarding scope of work details and method of payment for design consultant projects:

- (a) The DGCE/A or DPM shall consult the Railroad to determine any special requirements that the Railroad may have in order for the Department's contractor or consultant to gain access onto Railroad property.
- (b) Information obtained shall include but not be limited to Railroad insurance requirements, ROE permit/agreement fees, copies of applicable permits, time required to process permit, required safety training, Railroad specifications, and if necessary, associated flagging costs and necessary arrangements for flagging protection.
- (c) The DGCE/A or DPM shall ensure that all applicable Railroad ROE requirements are contained within the Department's scope of work/contract for the project.
- (d) The scope of work shall detail the method of payment for design consultant projects. If the method of payment is not included in the original consultant agreement, it shall be processed as a supplement.
- (e) For District in-house projects where geotechnical explorations are to be performed by Department personnel, the DPM or District Geotechnical Engineer (DGE) shall notify the DGCE/A so as to obtain all necessary Railroad ROE requirements and procedures, and method of payment prior to entry onto Railroad property. Note that the Department cannot sign Railroad ROE permits/agreements, but if the Railroad requires a permit/agreement to be entered into with the Department, consult with the Department's Office of Chief Counsel before proceeding further.
- (f) The DPM, DGE or DGCE/A shall make provisions for reimbursement of Railroad costs based on Department guidelines and procedures.

6.3.3.3 Consultant/Contractor Right-of-Entry Requirements and Method of Payment

The following activities may be required by the Department's consultant or contractor, including any subconsultants or subcontractors, (Applicant) in order to obtain a Temporary Railroad ROE permit/agreement and method of payment:

- (a) Contact Railroad to determine any special requirements the Railroad may have.
- (b) The consultant's technical and price proposal shall address all procedures required by the Railroad in order to obtain the necessary Temporary Railroad ROE permit/agreement and method of payment for Railroad invoices, permit fees, etc.. This shall include, but not be limited to, ROE permit/agreement procedures with associated costs with Railroad turnaround time for processing permits/agreements, Railroad insurance, flagging costs, Railroad safety training, and required notifications to Railroad prior to entry onto Railroad property.
- (c) Information for each individual entry by Applicant shall be supplied to the Railroad in accordance with their specification and/or permit requirements when applying for the ROE permit. This shall include scope of work, exact location of the entry, estimated cost of work to be performed on the Railroad property, distance from the outermost track where the activities will take place, anticipated date of entry, number of people and length of time.
- (d) The Applicant shall obtain a copy of the Railroad's Temporary ROE permit/agreement. The Applicant shall submit the completed and signed permit/agreement, with applicable supporting documentation and permit fees to the Railroad for execution. The Railroad shall forward the executed ROE permit/agreement to the Applicant. The Railroad will advise the Applicant of any necessary modifications required based upon the Railroad's policies and procedures.
- (e) Prior to entry onto Railroad property, an executed ROE permit/agreement must be in the possession of the Applicant.
- (f) The Railroad will submit invoices directly to the Applicant for services rendered and payment shall be made by the Applicant in accordance with the Railroad's procedures.

6.3.3.4 Entry onto Railroad Property

Prior to entry onto the Railroad property for the geotechnical exploration, the following activities are required:

- (a) Determine an exact scheduled starting date and ending date for the entry onto Railroad property.
- (b) Incorporate these dates in the letter seeking bidder interest and the proposal.
- (c) Where District policy allows an alternative to a defined schedule, note the desired method of scheduling in the letter seeking bidder interest and in the proposal.
- (d) Notify the District of the scheduled starting date at least thirty (30) days before the start date and provide the Railroad the required notice in accordance with the Railroad's specifications and/or permit requirements.
- (e) Comply with the Railroad's required safety training prior to entering.
- (f) Maintain an independent record of personnel making entry, date of entry, purpose of entry, Railroad personnel and equipment present and their activities, etc.. Review and countersign Railroad time sheets(s) daily; do not countersign if there is a disagreement - notify the District. Forward a copy of the time sheet(s) to the District within fifteen (15) days of their receipt.

6.3.4 Preparation and Management of Subsurface Boring, Sampling and Testing (SBST) Contract

6.3.4.1 Priorities and Objectives

The PGM should remain aware of the overall objectives of the SBST contract, and make appropriate efforts to ensure that the following priorities and objectives are met:

- (a) Obtain adequate information to establish all design parameters with sufficient reliability and to address all subsurface problems that may arise in design or construction.
- (b) Ensure that the following Department personnel (as appropriate) are promptly informed of any conditions that may result in a change in the contract, or any unusual conditions that are encountered during exploration:
 - (1) District Geotechnical Engineer
 - (2) District Bridge Engineer (or Plans Engineer if no structure work is involved)
 - (3) District Environmental Manager
 - (4) Project Manager
- (c) The subsurface conditions determined by the operations are fully and accurately described in the logs and records.

6.3.4.2 Contract Management

Projects drilled under Department contracts will be inspected by District personnel or the District's representative. Projects drilled under Consultant design contracts will normally be inspected by the Consultant's personnel.

The review of all inspections will be the responsibility of the Project Geotechnical Manager (P.E. or P.G.).

One full-time, certified inspector shall be assigned to each operating drill rig. Supervision shall be provided in accordance with Publication 222. A supervisor shall not be permitted to assume the duties of an inspector, except in an emergency.

The District Engineer shall modify the above only in exceptional cases.

Any supervisor or inspector who does not demonstrate proficiency or dependability at the project site, or engages in activities contrary to the best interest of the Department as determined by the District Geotechnical Engineer, will be subject to immediate removal. A qualified replacement will be required before work will be permitted to resume.

6.3.4.3 Preparation of Proposal/Contract

A proposal/contract for undertaking the necessary field work shall be prepared using the current Subsurface Boring, Sampling and Testing Contract in accordance with the Geotechnical Investigation Manual (Publication 222) after the draft copy has been accepted and when directed.

Contracts let by the Department shall be advertised in the Pennsylvania Bulletin in addition to the following procedure:

- (a) Send notice to all affected property owners as detailed in Publication 222, Subchapter 5E, Section 103.11.
- (b) Consider entry onto railroad right-of-way.
- (c) The requirements for working near overhead utilities are discussed in Section 103.09(b) of the Standard Specifications for SBST in Publication 222. When any part of the drill rig, including drill steel, is proposed to be located within 6 m (20 feet) of overhead power lines, the PGM/DGE shall contact the utility owner to determine the voltage in the line in order to ensure that the clearance distance requirement of Section 103.09(b) is met. In situations where clearance between overhead lines and drill equipment is less than required in Section 103.09(b), the PGM/DGE must coordinate this work with the appropriate utility company. Specific directions and necessary special provision(s) and pay item(s) to conduct this work must be prepared by the PGM/DGE and included in the contract.
- (d) The requirements for working where methane gas or carbon monoxide gas may be encountered are discussed in Section 103.09(c) of the Standard Specifications for SBST in Publication 222. In situations where it is suspected that methane or carbon monoxide gas may be encountered (e.g., where deep mines are likely to be encountered, where drilling in the vicinity of deep mines, or where mineable coal seams may be encountered) the PGM/DGE must include specific direction for methane and carbon monoxide monitoring in the contract. Typically, monitoring requirements are included in the schedule of drilling, and monitoring is incidental to the cost of drilling; however, special provisions or pay items

may be warranted in certain situations where an atypical level of monitoring is needed. During the subsurface investigation, if an elevated concentration of methane or carbon monoxide is encountered which prevents drilling and grouting of boreholes, the PGM/DGE should be prepared to provide direction to the drilling contractor. The PGM/DGE may contact PA-DEP personnel, the CGE, and/or others for technical assistance.

- (e) The PGE shall consider if a specific method of advancing soil borings is required or preferred, based on the anticipated site conditions. SBST Contract document Attachment I, Schedule of Proposed Borings, should typically indicate the anticipated method of hole advance for soil borings will be “Advanced by means selected by Contractor”. By doing this, the Contractor is responsible to select the most effective means/methods to accomplish the work. If the method does not work he will have to modify his methods. If, when preparing the contract, the PGE decides a specific method of boring advancement is needed, that method should be specified (i.e., hollow-stem auger, drive casing). Hollow-stem augers are the commonly preferred choice of boring advancement; however other options such as steel casing are occasionally employed. In some cases, augers cannot successfully advance – like cobbles/boulders, fills with obstructions, very dense tills, limestone ledges, cap rock above mine voids, etc.
- (f) Prepare the SBST Contract documents in accordance with the investigation plan approved by the Department. Prepare a plan of the foundation layout indicating the location of the test borings and other pertinent information.
- (g) Prepare an Engineers Estimate of the cost of the work.
- (h) Prepare a draft letter of interest advertising solicitation of bids for the subsurface investigation contract. Inform potential bidders in the letter that a reply is required by the date indicated in the letter of interest, and advise that all replies must be sent back to the email address indicated using the Letter-of-Interest Response Form supplied in the email. Except for cases as noted in Section 4.5 of Publication 222, the letter is to be sent by email to all Drilling Contractors on the current list of PennDOT Prequalified Drilling Contractors. Include a copy of the Letter of Interest and the Letter-of-Interest Response Form with the email to each prequalified Drilling Contractor. Include the following in the letter:
 - (1) Provide the anticipated type of borings, samples, and field tests with approximate quantities.
 - (2) Provide the location of the holes (water or land).
 - (3) State whether or not Maintenance and Protection of Traffic drawings will be provided or required.
 - (4) State whether any boring or test pit locations are within 30 m (100 feet) of any domestic water supply well or spring box, and if so include the appropriate as required by Section 219 of Publication 222.
 - (5) State whether the railroad requires safety training prior to entry on its right-of-way; anticipated dates for bid opening and Notice to Proceed.
 - (6) Attach a map that would allow anyone unfamiliar with the area to find the project.
 - (7) Describe any other pertinent information.
- (i) Prepare a draft of the letter to be used to transmit the SBST Contract documents to the Drilling Contractors who respond to item (h). Include in the letter the county, state route, and section numbers of the project; the date, time and location of the mandatory pre-bid meeting (if required); the date, time and location of the bid opening; and a reminder to follow the instructions in the Instructions to Bidders.
- (j) Prepare and transmit the contract documents to the drilling contractors expressing interest. The cover letter and accompanying contract documents must be sent by way of certified mail or hand-delivery.
- (k) Send three sets of the items outlined in Items (f) through (i) to the District Office for review and concurrence by the District Geotechnical Engineer.
- (l) Send the letter requesting interest (Item (h)) upon receipt of concurrence from the District.

6.3.4.4 Facilitation of Contract Bidding

The PGM shall ensure that the following requirements are completed during the bid process:

- (a) Conduct the mandatory pre-bid meeting unless indicated otherwise by the DGE. At this time all borings shall be staked on the project. Pre-bid meeting minutes must be sent to all contractors that attended the pre-bid meeting. Minutes may be sent via certified mail, email (with read receipt verification), or hand delivered). Sufficient time must be given to allow contractors to receive and review pre-bid meeting minutes prior to bidding date.
- (b) Attend the bid opening at the project District office, if practical. Alternately, bids may be opened at the consultant engineer's office, provided that the DGE or representative is present. Check the bid documents for completeness, correctness of quantities, and cost tabulations. Tabulate the bid results. Determine the low bidder. Obtain concurrence of the District Office in writing on the acceptability of the low bidder.
- (c) Notify all bidders of the results; send each a tabulation of the bids. Return all proposal guarantees except those of the two lowest bidders. Notify the lowest bidder to complete the forms in the bid package and furnish the required Contract Bond, Additional Bond for labor and Material, and insurance certificates. Notify the second lowest bidder that his/her proposal guarantee will be held until the lowest bidder completes the necessary paperwork.
- (d) Review the lowest bidder's submissions for completeness. The insurance certificate shall name the Commonwealth of Pennsylvania as co-insured. When all submissions are complete, return the proposal guarantees and issue a Notice to Proceed. Notice to Proceed should clearly state the day on which the time charges will begin.
- (e) Provide the DGE a copy of all correspondence at the time it is sent out. Only one copy of the advertisement letter is needed. Furnish a copy of all pages in the bidding package in which the contractor has filled in information, and copies of the bonds and insurance certificates.
- (f) Contracts let by the Department shall be advertised in either the Engineering and Construction Management System (ECMS); or the Supplier Relationship Management (SRM) system, as appropriate.
- (g) Upon award of contract to successful bidder the PGM (Consultant for consultant contracts, DGE for department contracts) must submit a completed copy of the "Form of Proposal" (Publication 222, Sub-Chapter 5C) to the CGE to the address below. This includes work performed under provisions of Section 4.5 of Publication 222. Include the completed copy of the Letter-of-Interest Response Form for the successful bidder (or entity performing work when using Section 4.5).

Chief Geotechnical Engineer
Innovation & Support Services Division
PennDOT Materials Testing Laboratory
81 Lab Lane
Harrisburg PA 17110-2543

6.3.4.5 Drilling Inspection Requirements

6.3.4.5.1 Minimum Requirements for Drilling Inspection

The following minimum qualifications must be met by all drilling Inspectors:

- (a) Speaks, writes, reads and understands the English language fluently.
- (b) Has prior drilling inspection experience on a minimum of three (3) drilling projects (Department or private), with minimum logging experience of 60 m (200 feet) of Standard Penetration Testing (footage from auger advance may not be included) and 30 m (100 feet) of rock coring.

- (c) Is knowledgeable of general drilling practices and the following references:
 - (1) Publication 213 (Temporary Traffic Control Guidelines)
 - (2) Publication 222 (Geotechnical Investigation Manual)
 - (3) Publication 293 (Geotechnical Engineering Manual)
 - (4) A copy of the boring contract for the project
 - (5) A rock identification text (e.g., the Audubon Society Field Guide to North American Rocks and Minerals, the Smithsonian Handbook of Rocks and Minerals, or other reference suitable for identification of PA rocks)
- (d) Be able to: traverse rough, steep terrain; cope with inclement weather conditions; deal with potential natural hazards such as snakes, ticks, bees and poison ivy; work safely in the vicinity of live traffic and open bodies of water.
- (e) Understands the general geotechnical design principles involved in the anticipated construction, and the anticipated laboratory testing.
- (f) Successfully passed both parts of the Department's Drilling Inspector Examination and possess a valid drilling inspector certification issued by the Department.
- (g) Capability of acting as a direct representative of the Engineer.
- (h) Demonstration of the above to the satisfaction of the District Geotechnical Engineer at an interview to be held at least two weeks prior to the start of the work. Names and qualifications shall be submitted prior to the interview.

6.3.4.5.2 Duties of Inspection Forces

The general duties of a drilling inspector include:

- Verify the testing and sampling procedures are performed correctly.
- Ensure the depth of each sample is known and recorded correctly.
- Describe soil and rock samples correctly.

Prior to the Start of Drilling Operations

The PGM is responsible to notify the DGE of the planned start-date of drilling and to forward to the DGE a copy of Drilling Contractor's submission providing information required by Chapter 5, Sections 104.01, 104.03, 104.08 and 104.09 of this publication.

The inspector is responsible to:

- (a) Have a copy of the "Intent to Enter" letter available at the site and provide information on all affected property owners to the Drilling Contractor (name, address, telephone number, etc.).
- (b) Determine the apparent adequacy of the Drilling Contractor's drilling equipment. If any equipment appears faulty or questionable, the inspector should discuss this with the driller and the PGM/PGE, if warranted.
- (c) Determine the adequacy of the Drilling Contractor's equipment for temporary maintenance and protection of traffic.
- (d) Determine if the Drilling Contractor has contacted all utilities and that all utilities are clearly marked in the field.
- (e) Visually confirm all plan locations of borings, test pits and instruments are clearly staked according to plans or instructions. Note if any locations appear to be within 100 feet of any domestic water supply wells or spring boxes and if so, confirm that the contractor has the proper provisions as specified in Section 219 of Publication 222.

- (f) Conduct field view of the site and understand what work is to be done. Note any environmental sensitive areas and if any temporary traffic control will be needed.
- (g) Study all documents, reports, and any other previous borings that were done at the site or nearby sites.
- (h) Have a list of emergency phone numbers, hospitals, and maps available at the site.
- (i) Study the drilling contract and all attachments, as applicable.
- (j) Be familiar with the subsurface investigation plan.
- (k) Understand the information to be gathered, its purpose in design and use for lab testing
- (l) Determine if the required safety courses have been taken by the workers (such as for railroad or hazardous waste work).

During Drilling Operations

The PGM/PGE is responsible to keep the DGE promptly informed of any issues or problems.
The inspector's responsibilities include:

- (a) Complete the Drilling Inspector's Daily Report.
- (b) Keep daily drilling quantity records/summaries, and review quantities with the driller each day work is completed.
- (c) Label all samples and core boxes.
- (d) Identify boring and test pit locations according to the plans or other instructions.
- (e) Ensure that the operations are performed according to the terms of the work and all relevant publications and guidelines.
- (f) Log test borings and test pits in accordance with Publication 222, Section 3.6.
- (g) Document instrument installations, including, but not limited to, types, locations, installation procedures and any problems encountered during installation.
- (h) Record any instructions given to the contractor including date, time, to whom, and the driller's response and whether or not actual work was performed as per the instructions.
- (i) Maintain any records required by this publication or the DGE.
- (j) Record in detail any events or situations that may later result in litigation or a claim against the Department.
- (k) Confirm and have the contractor make necessary field adjustments of the work zone traffic control set-up using PennDOT Publication 213, Temporary Traffic Control Guidelines.
- (l) Act as a responsible representative of the Department at all times.
- (m) Perform other related duties as directed or required.
- (n) Promptly report any unexpected findings or problems to the PGM/PGE.
- (o) Have the following available at the jobsite:
 - (1) All required technical references (Publication 222, Section 3.1, 3rd bullet) and other pertinent geologic references
 - (2) Driver's license

- (3) Black pen, pencil, eraser, calculator
- (4) 10% HCl acid solution
- (5) Pocket knife, hand lens, common hardwood dowel (not oak), copper pipe or penny, common steel nail, steel file,
- (6) PennDOT Drilling Inspector Certification Number
- (p) Select locations and depths for sampling and in-situ testing.
- (q) Record changes such as minor shifts in location, depth of boreholes, etc.
- (r) Review and countersign the daily time sheets connected with work on the Railroad property.

6.3.4.5.3 Field Logs and Engineer's Logs

The Inspector is required to log all borings in the field, observing and recording information as the boring is being completed. The Inspector's field logs are to be submitted daily or weekly as required by the PGM/PGE or DGE. Log a boring by recording the field observations needed to complete the final standard format "Engineer's Log", including information identified in Section 6.3.4.5.3a and 6.3.4.5.3b. Log a test pit by recording the information required on the final standard format "Engineers Test Pit Log" including the applicable information identified in 6.3.4.5.3c. Describe soil samples and rock core samples in conformance with the requirements of Publication 222, and record on standard hand-written field logs or PDA-generated field logs.

6.3.4.5.3a Information Required on the Inspector's Field Log

- (a) Project identification, including state route, section and county
- (b) Identification number for the test boring or test pit
- (c) Date on which the boring was begun. Date and time of completion of boring.
- (d) Name of the drilling company and drill operator
- (e) Drilling Inspector name and PennDOT certification number (for District and Consultant inspection)
- (f) Location of test boring relative to project reference line (e.g., segment/offset or station/offset from centerline) or other suitable references given by the PGE
- (g) Type of drill rig used
- (h) Drilling method used to advance the boring in soil
- (i) Size of soil sampler
- (j) SPT hammer type (automatic-trip, safety, or donut). Record the measured hammer efficiency rating (ER), if known.
- (k) Casing type, diameter and depth (if used)
- (l) Size of hammer and free fall used to advance casing (if used)
- (m) Drilling method used to advance the boring in rock
- (n) Type and size of core barrel used. Include condition of the core bit, if known.
- (o) Change in drilling method and reasons for such changes
- (p) Depth, type, number, relative moisture content and recovery of each soil sample

- (q) Hammer blows to advance split barrel sampler 15 cm (6 inches). If sampler advances less than 15 cm (6 inches) after 50 blows, record the depth of penetration for 50 blows (i.e., 0.2 ft / 50).
- (r) Relative density of cohesionless soils based on SPT
- (s) Consistency of fine-grained cohesive soils based on SPT
- (t) Unconfined compressive strength of fine-grained cohesive soils based on Pocket Penetrometer and/or Torvane tests
- (u) Depth, type, number, and length of core recovered, Rock Quality Designation (RQD) and percent recovery for each run of rock core
- (v) Description and identification of each soil and rock stratum as indicated in Section 3.6.3, Section 3.6.4, and Section 3.6.5.
- (w) Depth to top and bottom of profile for each distinctive interval, layer or horizon in soil, rock and anomalous zones
- (x) Depth to groundwater level, measured immediately after completion of drilling (0-hour), and 24-hours after completion of drilling. Record method by which observation was made.
- (y) Depths at which field tests are made and results of the tests (if available)
- (z) Difficulties in drilling such as obstructions, caving, boulders, rising of sand into bottom of boring, etc. Include basis for unusual or low recovery. Low recovery is generally considered REC<90% in rock, and REC<70% in soil.
- (aa) Depth of loss and return of circulating water. Also note any increase in usage of drilling water
- (bb) Any additional information such as changes in the color of drill return water, tool drops, drilling advancement rate, etc., which may be of assistance in defining the presence of strata changes, boulders, voids, or other subsurface conditions. Any other conditions or situations encountered during drilling such as loss of soil sample or rock core, water conditions, etc., that may be useful in assessing subsurface conditions.
- (cc) Elevation of top of test boring.
- (dd) Basis for low (less than 70% in soil and 90% in rock) sample recovery.
- (ee) Blows per 1 ft. to advance casing (optional if driven).

Standard Inspector's Field Log is shown in Figures 3.6(j) – 3.6(l) of Publication 222.

6.3.4.5.3b Information Required on the Engineer's Log

The Engineer's Log must be in the format shown in Figure 3.6(i) of Publication 222. The final log is comprised mostly of the information taken from the Inspector's log, but may also contain appropriate information from the laboratory testing results and the Driller's log as determined by the PGM or PGE of record.

The Engineer's Log must be signed and sealed by the PGM or PGE of record, attesting to the accuracy of all information on the log. A Professional Engineer (P.E.) seal or Professional Geologist (P.G.) seal is required. If someone assisted the PGE/PGM in the review of the field log and/or final log, their name and review date shall be recorded on the log.

In addition to the information recorded on the Inspector's field log, the final Engineer's Log is to include the following:

- (a) Elevation of top of test boring to the nearest 3 cm (0.1 foot).
- (b) Elevation of top of rock to the nearest 3 cm (0.1 ft), if applicable

- (c) Coordinates of the boring: The Inspector is to record the horizontal coordinate of each boring. This information will be provided by the project designer and will be in one of the following formats:

State Plane Coordinates: _____ . ____ N _____ . ____ E

Geodetic Coordinates: N ____ ° ____ ' ____ . ____ " W ____ ° ____ ' ____ . ____ "

Although State Plane Coordinates (northing and easting) are commonly used in project surveying, Geodetic Coordinates (latitude and longitude) of all borings are to be provided by the project designer and must be recorded on the final boring logs. This is required to enable standardized electronic data entry and also to facilitate the long-term retracement of boring locations.

- (d) Graphic column that gives a visual depiction of different soil and rock types encountered in the boring. This shall use the patterns shown in Figure 3.6(m) and also accent shading shown in Figure 3.6(o) of Publication 222 to assist in the clarity of the graphics.
- (e) Any data from laboratory testing directed by the PGM/PGE, such as AASHTO soil classification, USCS soil classification, gradation, Atterberg limits, and water content.
- (f) SPT Hammer Efficiency. SPT Hammer energy efficiency can be measured as specified in ASTM D4633, Standard Test Method for Energy Measurement for Dynamic Penetrometers. The Department requires the drilling contractor to provide measured hammer efficiency (ER_m) values for Donut hammers. The Department is currently not requiring measured ER_m values for Auto-trip hammers or Safety hammers; however, this information is to be recorded on the log if available from the driller. If actual calibration data is not available, an assumed hammer efficiency value (ER_a) will be recorded on the Engineer's Log and noted as such. The corrected N-value will be calculated and recorded on the Engineer's Log using the following equation:

$$N_{60} = \frac{ER \times N_{unc}}{0.6}$$

where:

N_{60} = corrected N-value (rounded down to the nearest whole number)

ER = hammer efficiency (1.0 maximum)

N_{unc} = uncorrected N-value, number of hammer blows on the bottom 300 mm (12 inches) of penetration

Assumed hammer efficiency values (ER_a) based on hammer type will be according to the following table. Note that for an assumed Safety hammer efficiency, no correction is necessary since the correction equation yields a value of 1.0.

Table 6.3.4.5.3b-1 – Assumed Hammer Efficiency Based on Hammer Type

Hammer Type	Assumed Efficiency (ER _a)
Automatic-trip	0.80
Safety driver	0.60
Donut	0.45

Reference: CALTRANS 59-910683 Drill Rig Hammer Evaluation

A Standard Engineer's Log is shown in Figure 3.6(i) of Publication 222.

6.3.4.5.3c Standard Engineer's Test Pit Log

The following information shall be provided on the Engineer's Test Pit Log:

- (a) Professional Engineer or Professional Geologist seal, signed and dated, of the project geotechnical manager (PGM) of record, attesting to the accuracy of all information on the log.
- (b) Project identification, including state route, section and county
- (c) Test pit identification number
- (d) Coordinates, northing and easting or latitude and longitude
- (e) Date of excavation
- (f) Name of Drilling Inspector (District's or Engineer's Representative)
- (g) Elevation of top of test pit
- (h) Location of the test pit relative to project reference line (e.g., segment, offset, and offset from centerline) or other suitable reference points
- (i) Type of and size of excavation equipment used
- (j) Ease of excavation
- (k) Caving of sides
- (l) Seepage amount and elevation
- (m) Depth to top and bottom of profile for each soil type
- (n) Depth, type and condition of rock (if encountered)
- (o) Relative density or consistency of soils
- (p) Pocket penetrometer or Torvane test results
- (q) Moisture condition of soils
- (r) Presence of boulders
- (s) Reason for stopping excavation (e.g., caving sides, difficult excavation, limit of reach of backhoe, etc.)
- (t) Included with the log, provide digital photographs (minimum 8-megapixel resolution, .jpg format) of each test pit, with location information (project number, station, test pit number, depths, etc.) clearly identified in photograph. A sufficient number of photographs shall be provided to clearly identify all details.
- (u) Type of shoring (if used).

A Standard Engineer's Test Pit Log is shown in Figure 3.6(m) of Publication 222.

6.3.4.6 Disposition of Soil and Rock Samples, and Project Records

Upon completion of drilling operations (or periodically during the drilling operations for larger projects), arrange for the contractor to deliver all samples and core boxes to the location designated in the SBST Contract (Publication 222, Subchapter 5A, Article H). If the designated location is not the District's storage facility, arrange for delivery to the District facility, after work (logging and lab testing) with the materials has been completed. Any delivery to a District facility requires a seventy-two (72) hour notice, and must be accompanied by a PennDOT Form No. TR-440, Chain of Custody for Subsurface Boring Sample Boxes.

Sample specimens selected for laboratory testing shall be removed from the core boxes prior to delivery to the District's long-term storage facility. Accessing specimens from core boxes which have already been placed in long-term storage should only be done under unusual circumstances and will require coordination with the DGE. In all cases, when a soil or rock sample is taken from a core box for testing, a durable spacer (e.g., glass sample jar or foam "swim noodle") shall be used to fill the space created. A written description of the sample and purpose for removal shall be placed in the glass jar, or securely fastened to the spacer.

6.3.4.7 Work Orders and Disputed Work

Consult with the District Geotechnical and Project Manager (if applicable) prior to submitting a work order and when encountering a dispute on the type of work.

Submit work orders to the DGE or Project Manager, in letter form, explaining the reason for the work; quantity, type, unit prices, and money value for each item of work; include any sketch that may be necessary to explain or locate the work. Also submit a letter from the contractor agreeing to the quantity, type, unit prices, and money value for each item; include any other data that may be needed to justify the price. Both the work order and the contractor's letter should mention any agreed to time extension. Do not inform the contractor to proceed with the work until receiving written authorization from the District Executive.

Provide documentation of any disputed work.

6.3.4.8 Submissions to the Department

The project Geotechnical Consultant is responsible to assure the delivery of the following information to the Department's DGE in which the project is located:

- (a) Provide a copy of the Drilling Inspector's Log within forty-eight (48) hours of obtaining the last water level reading. These may be hand-written or computer-generated logs.
- (b) Provide a copy of the final type-written Engineer's Logs for test borings and test pits, and all field and laboratory tests, within fourteen (14) days of completing the investigation. Provide both paper and electronic versions. Electronic files must be in .pdf format on CD(s), DVD(s), or via eftp site, as directed and agreed to by the DGE. Paper logs and test results must be individually sealed by a P.E. or P.G., with signature and date, attesting to the accuracy of all information. The P.E. or P.G. must be the PGM or PGE of record and must be registered in the Commonwealth of Pennsylvania.
- (c) Provide an electronic copy of all core box and test pit photographs within fourteen (14) days of completing the project. Submit photo files in .png format on CD(s), DVD(s), or via eftp site, as agreed to by the DGE. Individually photograph each completed core box, providing a legible and clear photo of the open box, with a tape measure or ruler laid along the top or bottom edge of the box. The image of the core box shall be taken from directly above the open box, at a distance that fills the frame of the photograph as much as possible.
- (d) The Consultant is responsible to forward a signed copy of the completed form TR-440, Chain of Custody for Subsurface Boring Sample Boxes, to the DGE within seven (7) days of delivery of boxes to the PennDOT storage facility. The Consultant shall attach to the TR-440 an itemized, signed listing of all core boxes delivered. Blank TR-440 forms can be obtained by contacting the DGE office.
- (e) Provide a copy of any correspondence, records, notes, tabular forms, etc. developed during the investigation, and any other information that the Department considers necessary for its records. Provide files on paper, CD(s), DVD(s), or via eftp site, as directed by the DGE.

(f) Geotechnical Information

In addition to the items listed in PP1.9.4.3, the following shall be included in the Foundation Report (Subsurface Conditions at Structure Locations):

- (1) Brief description of each site including history, surface features, geological formation and items identified during the final exploration plan meeting
- (2) Typed logs, plan showing test boring layout and log plans prepared by the Geotechnical/Foundation Engineer
- (3) Soil and rock parameters to be used in design, with calculations and references
- (4) Results of the professional engineer certified laboratory tests
- (5) Discussions of the applicability of various foundation types and cost comparisons; recommendations for foundation substrata improvements
- (6) Other geotechnical information deemed necessary to help justify the foundation-type selected

The foundation submissions shall be made as detailed in PP1.9.4.

The Consultant shall submit the following electronic files on CD(s) or DVD(s) to the Department's CGE within fourteen (14) days of completion of the investigation, testing, or report:

- (a) Include all Engineer's Logs, field testing results, and laboratory testing results. Files must be in .pdf format.
- (b) Include all test pit and core box photographs. Files must be in .png format.
- (c) Include paper cover letter and electronic file on disk that lists the full contents of the disk(s). Letter must be in .doc or .pdf format. Letter must be sealed by a P.E. or P.G., with signature and date, attesting to the accuracy of all information. The P.E. or P.G. must be the PGM or PGE of record and must be a licensed professional engineer or licensed professional geologist in the Commonwealth of Pennsylvania.

6.3.5 Prequalification for Drilling Contractors

The prequalification criteria, submittals and approval procedures shall be as presented in Chapter 1 – Drilling Contractor Prequalification of Publication 222.

6.3.5.1 Prequalification Approval

The Geotechnical Section of the Innovation and Support Services Division of the BOPD is responsible for maintaining and periodically distributing a list of prequalified drilling contractors. A Prequalification Request Approval Committee, composed of at least three District Geotechnical Engineers and chaired by a designated individual from the BOPD, shall be responsible for prequalifying drilling contractors. The membership of the District Geotechnical Engineers shall be on a rotational basis.

6.3.5.2 Performance Evaluation

Within three months after each drilling contract is completed, the District Geotechnical Engineer shall prepare, review with the driller and send a completed performance evaluation report to the Geotechnical Section, Innovation and Support Services Division, BOPD.

6.4 SERVICE PURCHASE TEST BORING CONTRACTS

The standard specifications for subsurface boring, sampling and testing as shown in the test boring contract document shall apply to all subsurface investigation work performed for the Department, including work performed under a service purchase contract. If changes are required for a specific job, they shall be handled in a similar manner to special provisions and included in Attachment II of the test boring contract. This attachment is referred to as Modifications and Additions to Standard Specifications for Subsurface Boring and Sampling. It is in this section that modifications to the specifications can be made (which might include changes of method of payment, deletion of sections, modification of work, etc.) to suit various situations which may arise.

REFERENCES

American Society for Testing and Materials, Annual Book of ASTM Standards, Volume 04.08, Soil and Rock Building Stones, 1987

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1

PART A: POLICIES AND PROCEDURES

CHAPTER 7 – DRAINAGE STRUCTURES, SCOUR AND CULVERTS

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7.1 HYDRAULICS

7.1.1 Hydraulic Design

For hydraulic design of drainage structures, including bridge waterways, see Design Manual, Part 2, Chapter 10, and the applicable directives.

7.1.2 Hydraulic Data

The following hydraulic data for structures draining an area 0.5 mi² or greater shall be shown on the drawings:

- (a) Drainage area (in square miles)
- (b) 100-year and other floods as required and specified in Design Manual, Part 2, Chapter 10, including:
 - (1) Magnitude (cfs)
 - (2) Frequency
 - (3) Pertinent water surface elevation from section immediately upstream of the structure
 - (4) Pertinent velocity inside the bridge opening

7.2 SCOUR

7.2.1 Scour Investigation

Scour investigations shall be completed for all drainage structures and retaining walls (both cast-in-place and mechanically stabilized earth (MSE)) along streams. This investigation shall be included with the foundation submission and H&H Report. The investigation shall contain scour calculations per PP7.2.2. The investigation shall also include site inspections, including inspection of nearby structures as necessary. The reference for scour investigation is the FHWA Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges", Fourth Edition, May 2001. In addition, Hydraulic Engineering Circular, No. 23 (HEC-23), "Bridge Scour and Stream Instability Countermeasures" contains useful information on the selection and design of measures to minimize the potential damage to bridges and other highway components at stream crossings. Please note that HEC-23 does not supersede Design Manual, Part 4 (DM-4). For example, "Design Guideline 8" discusses an approach to design of riprap which is incompatible with DM-4; therefore, it cannot be used. Other sections of HEC-23 may be used to supplement the technical guidance in DM-4. Additional scour investigation references are included in PP7.2.6.

Spread footings on erodible material shall be considered only if scour calculations are completed and can be corroborated by a site inspection by the performance of spread footings in nearby structures which have survived major floods, or if properly designed protective measures are provided. Otherwise, the bridge foundation should be extended to sound bedrock or supported on piles. If a foundation is supported on piles, the pile design must account for the estimated depth of scour and include a check of column strength for the unsupported length.

For spread footings set below scour depth, backfill the excavation with durable rock riprap protection (riprap). Where history of the bridge site indicates that the channel becomes restricted due to accumulation of debris or ice, consider the constricted opening in the scour investigation. Where the maximum highwater elevation at the drainage structure site is due to a backwater condition resulting from the stage of a downstream waterbody, the scour investigation shall consider the calculations based on a 100-year flood resulting from the watershed upstream from the drainage structure site assuming no backwater from a downstream confluence.

Where dams exist upstream from the drainage structure site, the design flood for the dam and its spillway shall be considered in the scour investigation.

No scour analysis for pipe or box culvert is required. Refer to BD-632M for scour protection details for box culverts. Refer to RC-30M for scour protection for pipes.

7.2.2 Scour Design Flood and Scour Computation

Total scour depth is comprised of:

- (1) aggradation or degradation,
- (2) contraction scour, and
- (3) local scour.

Footings are to be designed based on the total scour depth obtained from a scour design flood. The scour design flood is defined as a 100-year flood, the flood of record (if available) or the overtopping flood (if less than the 100 year flood), whichever results in the worst-case scour condition. For temporary structures, use the scour design flood as specified in PP5.5.3.4.3.g to determine the scour depths.

The D_{50} is taken as an average of the streambed material size, in the reach of the stream, upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally, this would be the bed material size in the upper 1 ft. of the streambed, which may capture the armor layer (i.e., larger, more uniform particles) of the stream, if present. Significantly underestimating the D_{50} value may provide overly conservative scour depths. Therefore, acceptable means to estimate D_{50} include:

- Visual inspection
 - Appropriate for all types of bed materials. Field tools (e.g., sand gauge card, gravelometer, wire screen, etc.) are readily available to assist the hydraulic engineer in streambed particle size determination.
 - Not applicable for the K4 factor used in the CSU Pier Equation
- Sieve analysis from volume/bulk samples
 - Required when the K4 factor used in the CSU Pier Equation is less than 1

The D_{50} should not be estimated from soil surveys or soil borings. For most Pennsylvania streams, soil borings will severely underestimate the size of the streambed material. When a boring is taken within the channel area, it will sample a small diameter core (2-4 inches) through the bed material, soils layers, and typically down to bedrock. The boring diameter will limit the D_{50} measurement, since any sediment size greater than the boring diameter will not be captured. Note, if the D_{100} particle size is less than the core diameter and the sample is taken in the stream channel, the soil borings may provide a reasonable D_{50} . Additionally, the soil boring locations are determined based on the substructure unit's (e.g., pier) location and are not representative of the streambed material within the entire channel section.

The scour due to lateral movement or shifting of the stream should also be evaluated for bridges on floodplains with a history of lateral movements of the stream from one side of the floodplain to the other through geologic time. For multi-span bridges, a scour prism plot (Chapter 8, HEC-18), which illustrates the total scour depth at any location in the bridge opening, and a site evaluation shall be included with the scour analysis in the H&H and Foundation Reports. The scour prism plot should use the scour depths that have considered both engineering judgment and also reduction of scour depth from riprap placement. Pier and abutment foundations shall be designed for the maximum total scour to account for channel and thalweg shifting.

Local scour depths for piers and unprotected abutments shall be calculated using equations that apply to the sites and design conditions. Engineering judgment shall be used to select a depth of foundation based on the computed scour depths from applicable equations and site history. If depth of foundation cannot be established based on engineering judgment, the highest or the lowest scour depth may be used, provided the designer can justify it based on the proven performance of an existing structure at the same location or nearby location on the same stream. The total scour depth for piers shall be the sum of the general contraction scour and the local scour, which includes both the pier scour and the vertical pressure flow pier scour depth. Local pier scour shall be calculated using the CSU equation (Chapter 6, HEC-18) for live-bed and clear-water conditions when the pier footing is not exposed to the flow. The pier width in the CSU equation shall be the pier width perpendicular to the flow direction for the frequency event being considered. When using a value less than 1 for the armoring correction factor (K4) in the CSU equation, the streambed gradation size requirements in HEC-18 must be satisfied and a sieve analysis of a streambed volume/bulk sample must be provided. The complex pier foundation equations (Chapter 6, HEC-18) shall be used to calculate local pier scour when the pier footing is exposed to the flow. Arneson's equation for scour should be used to calculate vertical contraction scour for pressure flow conditions at a structure and added to the local pier scour depth.

Properly designed riprap and/or guide banks for abutment protection may negate the need to compute local scour depth for abutments. For piers, a scour depth reduction factor of up to 50% may be used for local scour when multi-layered riprap scour protection are provided as per Figures PP7.2.4-1, PP7.2.4-3 and PP7.2.4-5, and appropriate inspection frequency is provided. Note that the local scour will include the sum of the pier scour and the vertical pressure flow pier scour, if applicable. This provision is not in accordance with HEC-18, and was negotiated with the FHWA based on past experience and conservative design practices. Multi-layered riprap scour protection means more than one layer of riprap with a minimum placement depth, D , comprised of one full layer (based on the nominal placement thickness indicated in Pub 408, Section 850) on top of the footing, with additional riprap placed to the bottom of footing as shown in the Figures in PP7.2.4.

7.2.3 Superflood

Footings shall be checked for a superflood (Q500), which is defined as a 500-year flood event. Scour calculations shall be performed for this event to determine if the Q500 scour depth is below the bottom of the footing elevation. Stability of deep foundations, including unsupported length of piles, must be analyzed with the maximum scour depth at the Service I limit state, as specified in D3.7.

7.2.4 Footing Location Guidelines

The following guidelines are provided as a minimum criteria for foundations in river environment:

(a) General

1. The minimum placement dimension, D , is the nominal placement thickness as per Publication 408, Section 850.
2. Adjacent streambed/ground elevation is considered to be proposed ground above the footing toe.
3. For substructure units above the design floodplain but within the 500-year floodplain, the bottom of footing may be placed per Section PP1.9.4.4(e), but scour calculations and riprap requirements must be met.
4. For substructures units completely out of the 500-year floodplain, the bottom of footing may be placed as if it is not in a river environment, but shall not be less than frost depth. See Section PP1.9.4.4(e).

(b) Spread footings

If scour calculations indicate scour depths terminating above bedrock and the top of rock is greater than 6 feet below adjacent streambed, the foundation location for scour may be treated as if it is on soils as per PP7.2.4(b)3.

1. On non-erodible sound bedrock:

The sound bedrock may be defined as rock mass with discontinuities that are tight or open less than 1/8 in. (the rock mass is at least of the quality with Rock Quality Designation (RQD) 70% and recovery 90%) such as non-erodible limestone or granite. For bridge replacement projects, the bedrock may be considered non-erodible if there is no observed scour of the bedrock at the existing bridge and $RQD > 50\%$.

If necessary, conduct additional test or re-evaluate the core boring data to confirm the soundness of the bedrock.

The bottom of the footing may be placed flush with the surface of the bedrock or keyed 6 in. to 1 ft. below the bedrock (neat cut around edge of the footing). The foundation report shall describe the determination of the footing placement. If the footing is keyed, sliding can be ignored. Also, clearly indicate on the plans or in the specification that blasting is not permitted for rock excavation for the footing. Concrete of the footing shall be placed against the vertical cut surface. See Figure 7.2.4-1 for riprap placement at piers and Figure 7.2.4-2 for riprap placement at abutments and wingwalls.

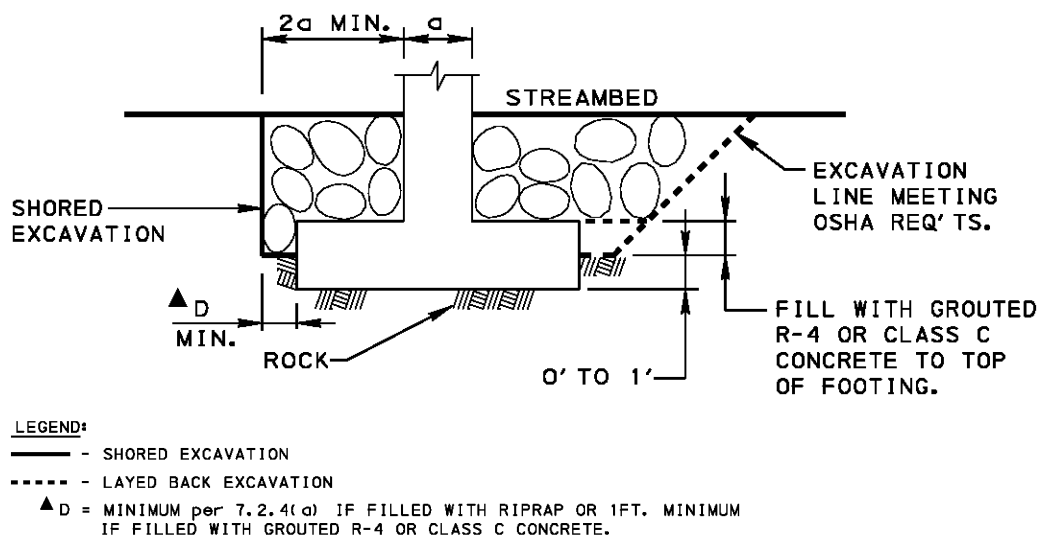


Figure 7.2.4-1 – Pier Footing on Non-Erodible Sound Rock

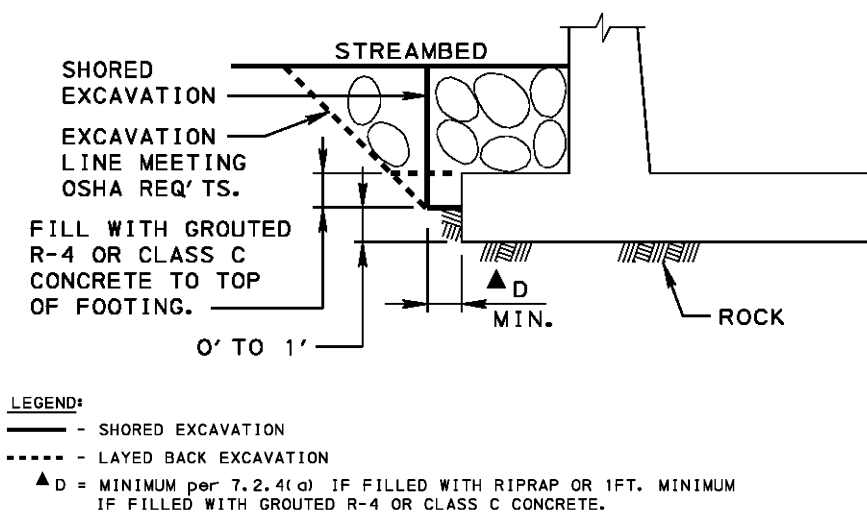
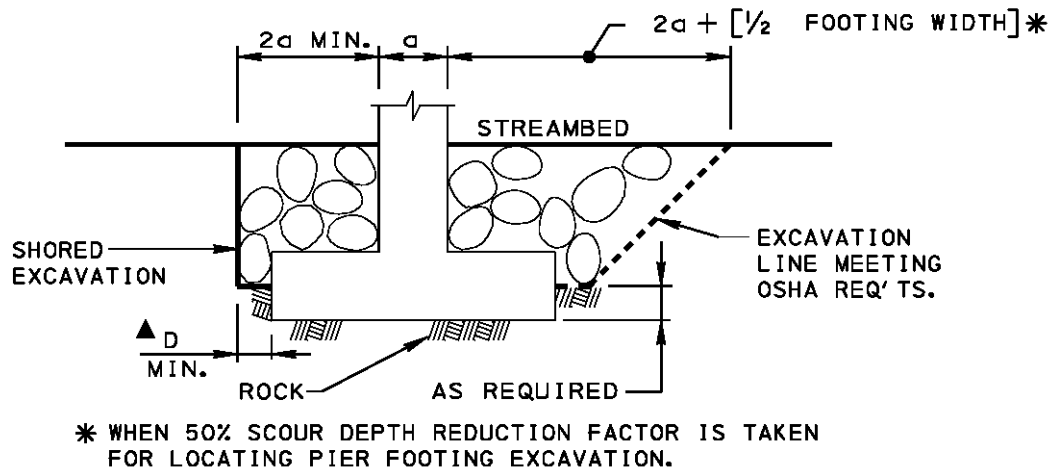


Figure 7.2.4-2 – Abutment/Wingwall Footing on Non-Erodible Sound Rock

2. On erodible rock:

Place the bottom of footing 3 ft. or more into the erodible rock even though the scour calculations indicate deeper scour for the scour design flood and superflood as specified in PP7.2.2 and PP7.2.3, respectively. If the calculated scour depth due to superflood is less than 3 ft. into the erodible rock, the bottom of the footing shall be located at the scour depth and a minimum of 6 ft. below the adjacent streambed/ground elevation. However, the bottom of footing need not be placed more than 3 feet into erodible rock to satisfy this provision. The bottom of footing elevation shall be confirmed by the engineering geologist who is familiar with the area geology and rock formations. This decision should be made based on an analysis of rock cores, including rock quality designation and local geology, as well as hydraulic data and past performance of nearby existing structures on the same stream, or in the case of a bridge replacement, the performance of the structure being replaced.

Blasting shall not be permitted to excavate rock for the footing. Concrete of the footing shall be placed against the excavated vertical surface. See Figure 7.2.4-3 for riprap placement at piers and Figure 7.2.4-4 for riprap placement at abutments and wingwalls.

**LEGEND:**

— SHORED EXCAVATION

----- LAYED BACK EXCAVATION

▲ D = MINIMUM per 7.2.4(a) IF FILLED WITH RIPRAP OR 1FT. MINIMUM IF FILLED WITH GROUTED R-4 OR CLASS C CONCRETE.

Figure 7.2.4-3 – Pier Footing on Erodible Sound Rock

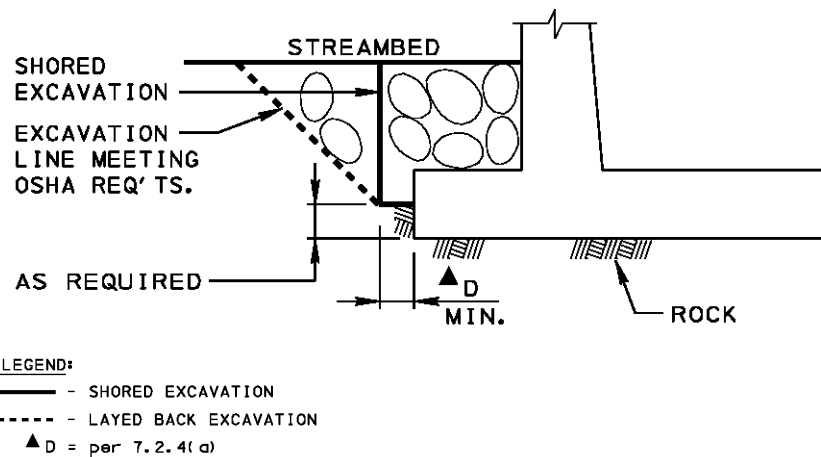


Figure 7.2.4-4 – Abutment/Wingwall Footing on Erodible Sound Rock

3. On soils including gravel, cobbles and boulders:

Place the top of the footing below the total scour depth for scour design flood as specified in PP7.2.2 with the bottom of footing not less than 6 ft. below the adjacent streambed/ground elevation.

Place durable riprap in all stream-side excavated areas in accordance with Figure 7.2.4-5 for riprap placement at piers and Figure 7.2.4-6 for riprap placement at abutments and wingwalls. Please note that additional riprap is required if the total scour depth based on superflood is lower than the bottom of footing elevation.

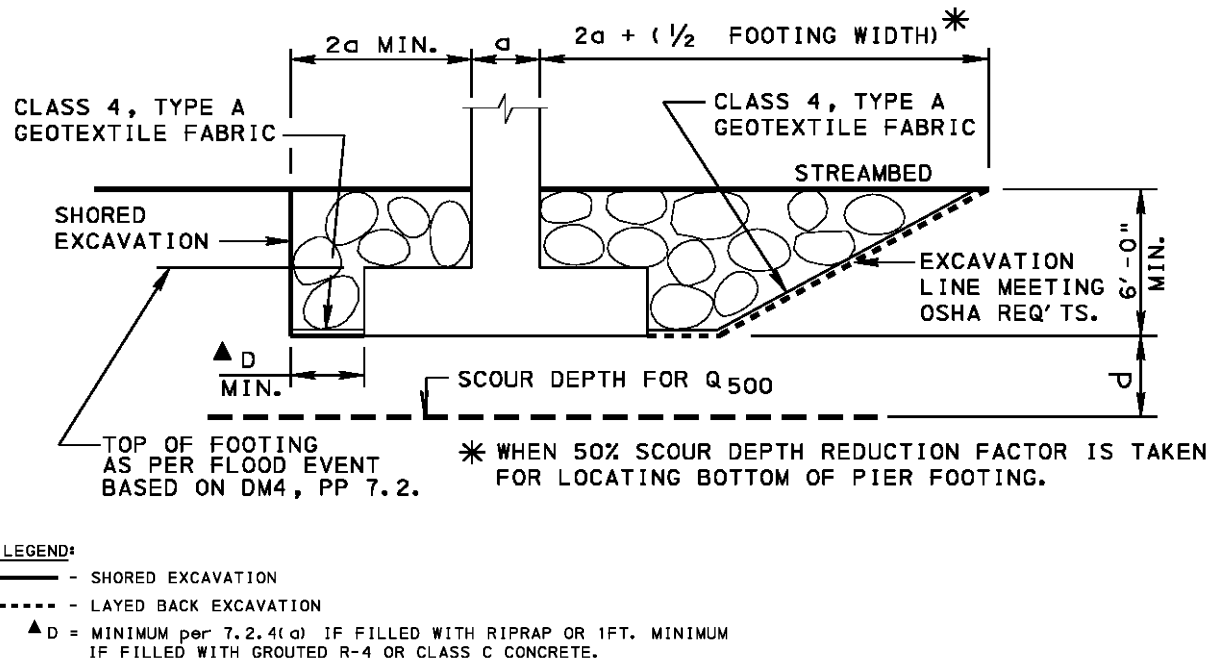


Figure 7.2.4-5 – Pier Footing on Soil

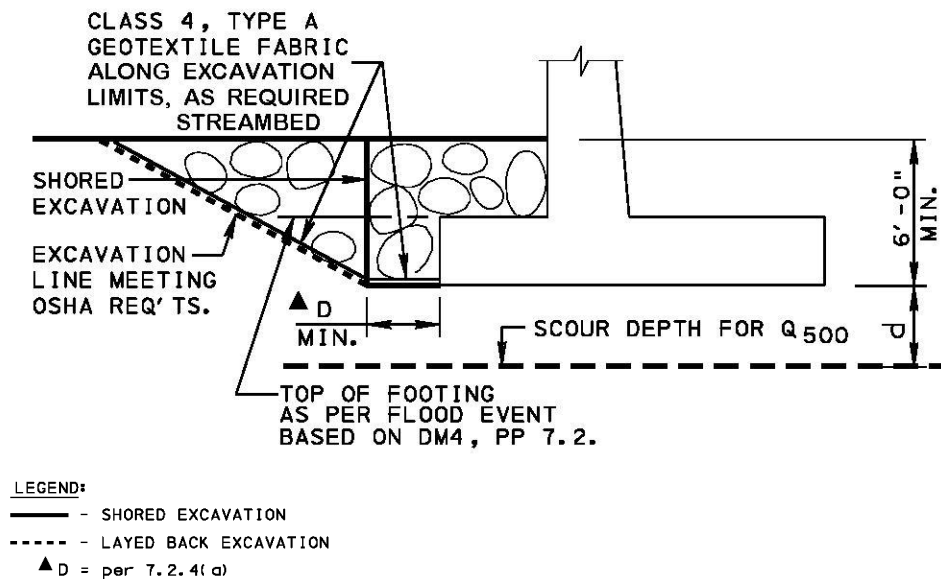


Figure 7.2.4-6 – Abutment/Wingwall Footing on Soil

(c) Footings on piles or drilled shafts

Place the top of the footing below streambed a depth equal to the estimated contraction scour (general scour) depth for scour design flood as specified in PP7.2.2. However, the bottom of the footing shall be located at least 6 ft. below the adjacent streambed/ground elevation for piers and arch abutments and at least 4 ft. below the adjacent streambed/ground elevation for abutments and wingwalls. For friction and end bearing piles, the pile length down to the total scour depth (including local scour) for the superflood shall not be considered in load transfer to the soils. For abutments properly protected with riprap or other approved means, piles may be assumed to be fully supported for unsupported column bending calculations. For piers, the scour depth reduction (credit) on local scour depths (due to riprap) shall be accounted for in determining unsupported pile length. Where special conditions indicate that the theoretical friction portion of a friction pile is substantially below the unsupported length due to scour, the top portion of the friction pile may need to be installed using auguring or other methods to preclude pile driving resistance in the unsupported length area during pile installation.

Piles and shafts in total scour zones shall be designed as laterally unsupported columns. Place the riprap similar to Figure 7.2.4-3 for piers. For abutments and wingwalls, place the riprap similar to Figure 7.2.4-4.

Special attention should be given to floodplains with a history of lateral movements of the stream from one side of the floodplain to the other through geologic time. In this situation, the worst scenario shall be used in locating the bottom of the footing.

7.2.5 A Guideline for Riprap Size Selection

Use Figure 7.2.5-1 in determining the riprap size. This figure is based on the FHWA formula $D_{50} = 0.692 V^2 / (S-1)(2g)$ for riprap sizing at piers, but may be used for all substructures by varying V based on the substructure location. For piers, V is taken as 1.5 times the average velocity at the section immediately upstream of the bridge. For vertical abutments, V shall be taken as 1.8 times the flow velocity in the bridge opening. The chart is prepared for determination of D_{50} for values of V up to 17.5 fps. For value of V higher than 17.5 fps, the above formula should be used. Use minimum riprap size of R-6.

The quality and gradation of the selected riprap should be as per Publication 408, Section 850. The D_{15-50} size listed in Section 850 shall be used as D_{50} for design purpose. For uniformity, use the following riprap size:

Table 7.2.5-1 – Riprap Size Selection

<u>V(fps)</u>	<u>Riprap Size</u>
up to 12	R-6 or larger
13 to 15	R-7 or larger
16 to 17.5	R-8

where V is 1.5 x velocity of flow for piers, or 1.8 x velocity of flow for abutments and wingwalls.

The velocity of flow should be based on the scour design flood specified in PP7.2.2. However, if the scour depth, based on Q_{500} , is lower than bottom of the footing elevation, then the velocity of the flow should be based on Q_{500} and not the scour design flood.

For values of V higher than 17.5 fps, special provisions should be developed to cover the size of the riprap by the above-mentioned formula.

In some special cases, the riprap size could be extremely large and impractical from Equation $D_{50} = 0.692 V^2 / (S-1)(2g)$. In these cases, design guidance for proper substructure protection may be found in the Hydraulic Engineering Circular (HEC) series.

The following guidelines are provided for riprap design:

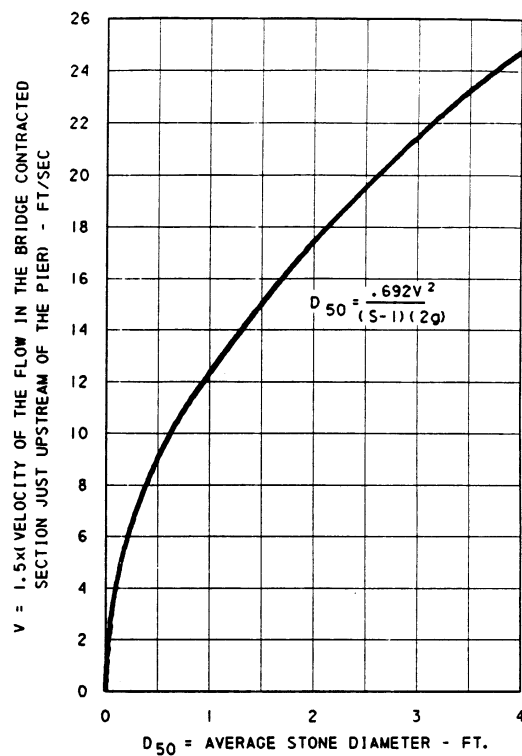
1. For practical design purposes, the maximum riprap size should be R-8. A larger riprap size may be used by the designer if available and required by design.

2. If a design dictates riprap size larger than R-8 and it is not available to the District or practical, the designer should consider other alternate designs such as:
 - a. A larger bridge opening which will reduce the riprap size due to the reduced velocity.
 - b. A pile supported or a caisson supported footing.
 - c. Guide banks or channel improvements.
 - d. Normal riprap with durable (estimated life of 50 years) woven metal fabric (e.g., gabions).
 - e. A cast concrete block (a manufactured concrete rock, not a concrete streambed paving) if no other method is suitable.

Provide a V-notch (3/4" minimum) in the stem of each substructure unit to locate the top of riprap protection. This is to help the bridge inspector in the field determine riprap loss. Consider choking R-8 and R-7 riprap with R-4 on the surface to provide better footing for the inspector. In environmentally sensitive areas, consider placement of natural soils above riprap. In these cases, the bottom of the footings may need to be lowered to ensure multiple layers of riprap protect the footing.

For riprap at piers, the lateral extent of riprap on each side of the pier face need not exceed twice the width of the pier.

Studies have shown that the local scour effects will not extend beyond a riprap mat of a total width of five times the pier width.



NOTE: S = SPECIFIC GRAVITY OF RIPRAP MATERIAL
(2.65 IN THIS CHART)
 g = 32.2 FT/SEC²

Figure 7.2.5-1 – D_{50} Rock Size

7.2.6 Scour Investigation References

Additional references for scour investigation are as follows:

- (a) Stream Stability at Highway Structures, Third Edition, Hydraulic Engineering Circular No. 20, FHWA Publication No. FHWA-NHI-01-002, March 2001.
- (b) Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1, Hydraulic Engineering Circular No. 23, FHWA Publication No. FHWA-NHI-09-111, September 2009.
- (c) Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 2, Hydraulic Engineering Circular No. 23, FHWA Publication No. FHWA-NHI-09-112, September 2009.
- (d) Evaluating Scour at Bridges, Fourth Edition, Hydraulic Engineering Circular No. 18, FHWA Publication No. FHWA-NHI-01-001, May 2001.

7.3 CULVERTS

7.3.1 Design Specifications

See A12 and D12.

7.3.2 Concrete Box Culvert Alternatives

The procedure regarding concrete box culvert alternatives shall be as follows:

- (a) A precast concrete box culvert shall be prepared as a primary design, with a cast-in-place (CIP) concrete box culvert as a contractor-designed alternate, unless it can be documented that a precast concrete culvert is not suitable for a specific site, e.g., non-uniform bearing strata, cost ineffectiveness. If hydraulically and structurally feasible, both precast and CIP concrete box culverts shall be allowed as alternates for a CIP concrete arch and other small steel or concrete bridges, unless documented environmental restrictions for natural streambed preservation control.
- (b) The entire culvert, wingwalls, apron and incidental excavation and backfill for the box, wings and aprons shall be included in the lump sum culvert item, except for reinforcement bars which are "AND Items". List culvert length in feet, excavation and backfill in cubic yards, and concrete quantities for wings and aprons in terms of cubic yards in the quantity table as information items and on the Component Item Schedule in the Proposal.
- (c) Provide riprap protection as per BD-632M.

7.3.3 Corrugated Metal Buried Structures

Corrugated metal buried structures may be used on local and collector roads with ADT of less than 750. This applies to both Federal and State-funded projects. Structural plate pipe and arches covered by BD-635M may be used for all highways, regardless of ADT, except where ADTT > 500. Chief Bridge Engineer approval is required for corrugated metal buried structures exceeding these limitations.

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PREFACE

In Part B, the AASHTO LRFD article numbering system is followed. Where a new article has been added, the suffix P, to designate "Pennsylvania Article", appear at the end of the new article number.

All references to AASHTO LRFD Bridge Design Specifications sections, articles, equations, figures or tables carry the prefix A.

References to AASHTO commentary carry the prefix AC.

References to Design Manual, Part 4, Part A, "Policies and Procedures", carry the prefix PP.

References to Design Manual, Part 4, Part B, "Design Specifications", carry the prefix D.

References to commentary to Design Manual, Part 4 carry the prefix DC.

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SPECIFICATIONS

COMMENTARY

1.1 SCOPE OF THE SPECIFICATIONS

The following shall supplement A1.1.

The design of any structure, or portion thereof, which is not covered by the DM-4, Part B, or the AASHTO LRFD Specifications shall be designed by other appropriate PennDOT or AASHTO documents as specified in PP1.1 or applicable Specifications and Standards approved by the Chief Bridge Engineer.

1.2 DEFINITIONS

The following shall supplement the definition of Design Life.

Design life is assumed to be 100 years for the main load carrying members of the superstructure and substructure. The design life of decks, barriers and expansion devices are considerably less.

1.3 DESIGN PHILOSOPHY**1.3.1 General**

The following shall supplement A1.3.1.

An elastic analysis shall be used to determine the force effects. The resistance of components and connections may be determined using inelastic behavior in the Extreme Event Limit State.

1.3.2 Limit States**1.3.2.1 General**

The following shall supplement A1.3.2.1.

In no case shall the value of η be less than 1.0 nor shall it be greater than 1.0.

When the Load Combination Extreme Event III and IV is investigated, the value of η shall be taken as 1.0.

1.3.3 Ductility

The following shall replace the third paragraph of A1.3.3.

For all components and for all limit states on all bridges, the ductility load modifier, η_D shall be taken as 1.00.

C1.3.2.1

The following shall supplement AC1.3.2.1.

In Pennsylvania, the application of load modifiers other than 1.0 has been found to have a detrimental affect on economical bridge design without a corresponding increase in comfort level.

C1.3.3

The following shall replace the last sentence of the second paragraph of AC1.3.3.

Such ductile performance shall be verified by testing and approved by the Chief Bridge Engineer.

The following shall replace the first sentence of the seventh paragraph of AC1.3.3.

The ductility capacity of structural components or connections may either be established by full or large scale testing, or with analytical models which are based on documented material behavior, but either method must be approved by the Chief Bridge Engineer.

SPECIFICATIONS

1.3.4 Redundancy

The following shall replace the second paragraph of A1.3.4.

For all bridges, the redundancy load modifier, η_R shall be taken as 1.00.

Global redundancy for truss bridges shall be provided by any of the following or combination thereof:

- provision of a third line of trusses where possible,
- use of stitched built-up components which are designed to support the entire component load with any one element assumed to be broken, and for which joints and splices have been designed to transmit component loads with any one element of the component assumed to be broken or,
- demonstration through 3-D analysis that failure of any tension component, or other components designated by the Department, of a two-truss system will not cause the collapse of the entire structure.

When the above provision for a stitched built-up component is invoked, the investigation shall be based on Load Combination Extreme Event III as specified in D3.4.1. When the above provision for 3-D analysis is invoked, the investigation shall be based on Load Combination Extreme Event IV as specified in D3.4.1. All live load models, specified in D3.6.1.2, shall be investigated. Approval by the Chief Bridge Engineer shall be required for all two truss systems.

The tie component of tied arched bridges shall meet the requirements of the above provision for a stitched built-up component.

Global redundancy for multiple element hangers composed of multiple bridge strands or multiple bridge ropes shall be provided by demonstration through 3-D analysis that failure of any tension component, or other components designated by the Department, will not cause the collapse of the entire structure. This analysis shall use Load Combination Extreme Event IV as specified in D3.4.1. All live load models, specified in D3.6.1.2, shall be investigated.

A two-girder bridge shall be used, only if (1) fracture-critical members are eliminated by developing alternative load paths and (2) approval is obtained from the Chief Bridge Engineer. The designer should evaluate the ability of secondary members to transfer loads and prevent collapse through the use of suitable computer analysis.

COMMENTARY

C1.3.4

Tied arched bridges will be considered globally redundant by meeting this requirement.

A two-girder bridge has the potential for fracture-critical members, i.e., a failure of one girder appears to cause collapse of the entire structure. There are certain situations, however, in which use of two girders may provide a significant economy.

For both two- and three-girder systems, computer-aided designs have been completed where secondary members are designed to transfer load around a failed portion of a girder, thereby preventing a collapse of the bridge.

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1.3.5 Operational Importance

The following shall replace the third paragraph of A1.3.5.

For all bridges, the operational importance load modifier, η_i , shall be taken as 1.0.

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2.3 LOCATION FEATURES**2.3.2 Bridge Site Arrangement****2.3.2.2 Traffic Safety***2.3.2.2.1 Protection of Structures*

The following shall supplement A2.3.2.2.1.

The minimum lateral clearance for locating substructure units for bridges over highways or railroads shall be as follows:

- (a) For bridges over a highway, refer to Design Manual Part 2, Chapter 4.4.C for the minimum horizontal clearance for locating substructure units.
- (b) For bridges over railroads, see D2.3.3.4.

2.3.2.2.2 Protection of Users

The following shall replace the last paragraph of A2.3.2.2.2.

When sidewalks are provided on bridges, with a posted vehicular speed greater than 45 mph or structures longer than 200 ft. (regardless of the speed), the sidewalk shall be protected by a barrier, unless waived by the Department.

2.3.2.2.3 Geometric Standards

The following shall replace the last sentence of A2.3.2.2.3.

For roadway geometry refer to Design Manual, Part 2. For bridge-mounted barriers, structure-mounted guide rail and other protective devices refer to appropriate Standard Drawings.

2.3.3 Clearances**2.3.3.2 Highway Vertical**

The following shall replace the first sentence of the first paragraph of A2.3.3.2.

Refer to Design Manual, Part 2, for minimum vertical clearance for overhead bridges.

The following shall supplement A2.3.3.2.

Vertical clearance over the width of the roadway, including shoulders, shall be provided in accordance with Design Manual, Part 2, Chapter 2, Section 2.20, and as

C2.3.2.2.2

The following shall replace the last paragraph of AC2.3.2.2.2.

An example where the Department may waive the sidewalk barrier requirement is a structure longer than 200 ft. in an urban environment where a curbed approach walkway exists and the posted vehicular speed is less than or equal to 45 mph. When a barrier is required on a bridge to protect the sidewalk, the barrier shall be transitioned to the appropriate roadway protection device (guide rail, barrier, curb, etc.) beyond the end of the structure and maintain pedestrian access.

For additional guidance on sidewalk barriers on bridges, refer to Design Manual, Part 2, Chapter 12.

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follows:

- For bridges over railroads, see D2.3.3.4.
- Minimum required vertical clearance shall preferably be maintained within the recovery area. In calculating actual vertical clearance under a beam splice, an allowance of 3/4 in., plus the thickness of the outside flange splice plate, shall be made.
- Vertical clearance is to be measured at any high point on the outer edge shoulder. 1 in. will be deducted from the actual measurement for posting purposes to account for minor variations as proper location of measurement and jumping of traveling vehicles.
- For vertical sag under the bridge, ensure that the vertical clearance is measured from a chord between any two high points along the traveling direction to account for the maximum truck length permitted on the road.

For prestressed concrete beams, do not take credit for beam camber in determining actual vertical clearance, unless the beam is cast or assembled specifically to provide vertical curvature of the bottom of the beam

2.3.3.3 Highway Horizontal

The following shall supplement the first paragraph of A2.3.3.3.

Refer to Design Manual, Part 2, for additional details on bridge widths, including criteria for bridges on Very Low Volume Roads.

2.3.3.4 Railroad Overpass

The following shall supplement A2.3.3.4.

Pennsylvania Public Utility Commission (PUC) has jurisdiction on railroad overpass clearances.

Refer to Design Manual, Part 1C, Chapter 4, Section 11, for additional details where railroads are overpassed by a highway structure.

Structures carrying railroad tracks shall be designed according to AREMA specifications and the modifications adopted by the railroad system involved.

For structures carrying highways over railroad tracks, the minimum horizontal clearance, specified and provided, from the centerline of track shall be in accordance with Publication 371 and/or Railroad Form D-4279 to face of abutment or pier and shall be shown on the drawings. An 18'-0" lateral clearance from the centerline of track shall be provided for off-track equipment on one side if requested by the railroad. Class 1 (major) railroads may require additional lateral clearance depending upon the need for drainage ditches and the roadway

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for off-track equipment. If track and abutment or piers are skewed relative to each other, horizontal clearances to the extremities of the structure shall also be shown. If the track is on a curve within 80 ft. of the crossing, additional horizontal clearance is required to compensate for the curve (refer to AREMA, Volume 2, Chapter 28). If a railroad requests clearance in excess of the above, complete justification of this request shall be provided. The agreement on the lateral and vertical clearances shall be reached with the operating railroad, or the determination from the PUC shall be secured prior to submitting for TS&L approval.

The minimum vertical clearance over the top of rail shall be in accordance with Publication 371 and/or Railroad Form D-4279 and shall be shown for each track on the drawings. If track and abutments or piers are skewed relative to each other, vertical clearances to the extremities of the structure should also be shown. Approval for any exception to the above minimum clearance over railroad tracks shall be secured from the operating railroad company or the PUC prior to submitting for TS&L approval.

To provide for a drainage ditch parallel to track, the elevation of the top of footings adjacent to track shall be at least 3'-6" below the elevation of the top of rail, unless rock is encountered.

The edge of footing shall be at least 7 ft. from the centerline of adjacent track.

If pier bents are used between 18 ft. and 25 ft. from the centerline of tracks, columns shall be protected by crash walls at least 2'-6" thick, which shall extend 10 ft. above the top of rail and 6 ft. for single column or 2'-6" for multi-column bents beyond the outside face of outside columns. The crash wall shall rest upon the column footings, extend 6 in. from the face of columns adjacent to traffic, and shall connect all columns in a pier bent. Solid piers with a minimum thickness of 2'-6" and a length of 20 ft., single column piers of minimum 4'-0" x 12'-6" dimensions or any solid pier sections with equivalent cross sections and minimum 2'-6" thickness negate the need to provide crash walls. Reinforcement to be designed in accordance with A3.6.5.2, but not less than horizontal bars of No. 6 at 12 in. each face, and vertical stirrups of No. 4 at 12 in.. Crash walls meeting the same dimension and reinforcement requirements as above shall also be provided in front of prefabricated walls.

Bridge scuppers shall not drain onto railroad tracks. Provision shall be made to direct surface water from the bridge area into an adequate drainage facility along the railroad track, in which case drainage approval by the railroad company is required prior to submission of final plans.

Safety provisions required during excavation in the vicinity of railroad tracks and substructures shall be in accordance with the Special Provision "Maintenance and Protection of Railroad Traffic".

Sheet piling used during excavation for protection of railroad tracks and substructure shall be designed according to AREMA specifications and shall be subject to approval by the

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railroad company. The use of caisson footings shall be evaluated in lieu of sheet piling and deep foundation.

Complete details of temporary track(s) or a temporary railroad bridge to be constructed by the Department's contractor shall be shown on the design drawings. Applicable railroad design standards or design drawings shall be referred to or duplicated on the design drawings.

For National Highway System (NHS) structures crossing over railroads, protective fencing in accordance with standard drawing BC-701M will be provided per the railroad's requirements (i.e. both sides, sidewalk only, etc.) for the portion of the structure (spans) over the railroad. For non-NHS structures with sidewalks, the protective fencing shall be provided only on the sidewalk side of the structure, for the portion of the structure (spans) over the railroad. For non-NHS structures crossing over railroads where protective fencing is not required by Department criteria, the railroad may request the installation of the protective fence for the portion of the structure (spans) over the railroad, if the railroad agrees to reimburse the Department for the installation of the protective fence.

All railroad clearances shall be based on the railroad's current design criteria.

For electrified railroad tracks, these additional requirements apply:

- If a railroad is electrified, it shall be so noted on the preliminary plans submitted for type, size and location approval.
- Protective barrier shall be provided on spans or on part of spans for structures over electrified railroads, as directed by the railroad company. Generally, the protective barrier shall extend at least 10 ft. beyond the point at which any electrified railroad wire passes under the bridge. However, in no case shall the end of the protective barrier be less than 10 ft. from the wire measured in a horizontal plane and normal to the wire outside of the limit of the bridge, and less than 6 ft. from the wire within the limit of the bridge.
- Details of protective barriers are shown on Standard Drawing BC-711M. If conditions warrant or if directed by the railroad company, details shall be modified. Such modifications shall be shown on the design drawings.
- All open or expansion joints in the concrete portion of barriers, divisors, sidewalks, and curbs within the limits of the barrier shall be covered or closed with joint materials. Details of such joints shall be shown on the design drawings.
- In the case of bridges crossing electrified railroad tracks, the details of catenary attachments and their locations, if attached or pertinent to the structure, shall be shown on the plans. Consideration shall be given to realign the catenary by installing support columns on each side of the

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bridge to avoid catenary attachments to the bridge. Normally, ground cable attachments, cables, miscellaneous materials, etc. are supplied by the contractor and are installed by the railroad. A separate block identifying materials required, description of materials, railroad reference number for materials, and party responsible for providing or installing materials shall be shown on the plans. Approval of grounding plans shall be obtained from the railroad concurrently with approval of the structure drawings.

Where the PUC has jurisdiction over the structure involved, the PUC Docket Number shall be shown on the first sheet of the design drawings (S-drawings) above the title block, after the PUC has approved the plans.

The responsible designer shall add the PUC Docket Number on all plans, and the District shall add the PUC Docket Number in BMS where the PUC has jurisdiction over the structure involved.

Where applicable, the USDOT/AAR number of the existing structure shall be shown on the first sheet of the design drawings.

2.5 DESIGN OBJECTIVES

2.5.2 Serviceability

2.5.2.2 Inspectability

D2.5.2.2.1P, D2.5.2.2.2P, D2.5.2.2.3P and D2.5.2.2.4P shall replace A2.5.2.2.

2.5.2.2.1P General

Inspection and maintenance instructions and requirements for critical details shall be stipulated on the plans. The plans shall also include reference to the method of access for inspection of the subject details.

It is necessary to have adequate means of access for bridge safety inspection. Review bridge designs for inspectability at TS&L, final design and construction stages. PP3.6.7 provides information for checking a structures inspectability with PennDOT's underbridge crane.

For special bridge conditions, the inspectability shall be as determined by the Chief Bridge Engineer.

2.5.2.2.2P Inspection Walks

2.5.2.2.2aP General

Unless approved otherwise, inspection walks shall be provided for long bridges (over 1,000 ft.) which cannot be readily inspected using inspection crane or which are otherwise inaccessible from underneath. Generally, inspection walks are

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required under the following conditions:

- Bridge width over 60 ft., inaccessible from underneath
- Superstructure depth over 11.5 ft., including beams, barrier, railing or fencing, and noise walls, inaccessible from underneath
- High bridge underclearance (in excess of 30 ft.), particularly for bridges over large rivers

2.5.2.2.2bP Design Live Load

A minimum design live load of 80 psf shall be used.

2.5.2.2.2cP Geometry

The minimum width shall be 4 ft.. The minimum overhead clearance shall be 6 ft.. Toe guard protection and railing shall be provided if the walk is not protected by the girders. The walk shall be secured against vandalism and shall not provide entrance to the general public. The entrances shall be locked or secured against access. Provision shall be made to cross from one bay to the next, generally at pier locations for at least one person, 6 ft. in height, carrying tools and equipment.

2.5.2.2.2dP Connection to the Main Members

Generally, a bolted or threaded insert type of connection shall be provided to secure the walks in position. Lock washers or another positive connection device shall be specified to protect the connection from being loosened due to bridge vibration.

2.5.2.2.3P Inspectability for Enclosed Section

Vent holes and large size (2 ft. by 3 ft. minimum and 3 ft. by 4 ft. desirable opening) inspection hatches shall be provided for large-span box structures; provision for lighting, cross ventilation, and steps shall be made where required. Large box sections shall have a coat of white paint on the interior.

2.5.2.2.4P Girder Handrail

Where other inspection facilities are not provided, handrails shall be attached to the web of steel girders greater than 72 in. in depth.

2.5.2.2.5P Sound Barriers

Sound barriers shall be designed and detailed to maintain bridge inspectability. For special conditions, the inspectability shall be determined by the Chief Bridge Engineer.

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2.5.2.6 Deformations*2.5.2.6.1 General*

The following shall replace the first and second paragraphs of A2.5.2.6.1.

Bridges shall be designed to avoid undesirable structural or psychological effects due to their deformations.

2.5.2.6.2 Criteria for Deflection

The following shall replace A2.5.2.6.2.

In applying criteria for deflection, the vehicular load shall include the dynamic load allowance.

To control deflections of structures, the following principles shall apply:

- when investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally,
- for curved steel box and I-girder systems, the deflection of each girder should be determined individually based on its response as part of a system,
- for composite design, the design cross-section should include the entire width of the roadway, neglecting any stiffness contribution by barrier, railing or other secondary members of the bridge,
- when investigating maximum relative displacements, the number and position of loaded lanes should be selected to provide the worst differential effect,
- the live load portion of Load Combination Service I of Table A3.4.1-1 should be used, including the dynamic load allowance, IM
- the live load shall be taken from D3.6.1.3.2,
- the provisions of A3.6.1.1.2 should apply,
- for skewed bridges, a normal cross-section may be used; for curved and curved skewed bridges a radial cross-section may be used.

The following deflection limits shall be used for steel, aluminum and/or concrete vehicular bridges:

- vehicular load, general..... Span/800
- vehicular and/or pedestrian loads Span/1000
- vehicular load on cantilever arms Span/300
- vehicular and/or pedestrian loads on cantilever arms..... Span/375

C2.5.2.6.1

The following shall replace the first, second and third paragraphs of AC2.5.2.6.1.

Service load deformations may cause deterioration of wearing surfaces and local cracking in concrete slabs and in metal bridges which could impair serviceability and durability, even if self limiting and not a potential source of collapse.

C2.5.2.6.2

The following shall replace AC2.5.2.6.2.

For a multi-beam bridge, this is equivalent to saying that the distribution factor for deflection is equal to the number of lanes divided by the number of beams.

For curved steel girder systems, the deflection limit is applied to each individual girder because the curvature causes each girder to deflect differently than the adjacent girder so that an average deflection has little meaning. For curved steel girder systems, the span used to compute the deflection limit should be taken as the arc girder length between bearings.

The weight of barrier, railing or other secondary members shall be included for deflection and design. Only the stiffness of these items should be neglected.

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For steel I-shaped beams and girders, the provisions of A6.10.4.2 and A6.11.4, respectively, regarding the control of permanent deflections through flange stress controls, shall apply. For pedestrian bridges, i.e., bridges whose primary function is to carry pedestrians, bicyclists, equestrians, and light maintenance vehicles, the provisions of Section 5 of AASHTO's LRFD Guide Specifications for the Design of Pedestrian Bridges shall apply.

The following deflection limits shall be used for wood construction:

- vehicular load, general..... Span/425
- vehicular and/or pedestrian loads Span/800
- vehicular load on wood planks and panels:
extreme relative deflection between
adjacent edges..... 1/8 in.

The following provisions shall apply to orthotropic plate decks:

- vehicular load on deck plate Span/300
- vehicular load on ribs of orthotropic metal
decks Span/1000
- vehicular load on ribs of orthotropic
metal decks: extreme relative deflection
between adjacent ribs..... 1/8 in.

2.5.2.6.3 *Optional Criteria for Span-to-Depth Ratios*

The following shall supplement the first paragraph of A2.5.2.6.3.

The span-to-depth ratios given in Table A2.5.2.6.3-1 should be used for preliminary sizing of structures or components of structures and are not mandatory limits. In all cases, the final structure or components of structures shall satisfy the deflection criteria of D2.5.2.6.2.

Concrete decks on multi-girder-type bridges shall satisfy the span-to-depth ratios in Table A2.5.2.6.3-1 with the heading "Slabs".

2.5.2.7 **Consideration of Future Widening**

2.5.2.7.1 *Exterior Beams on Multi-Beam Bridges*

The following shall replace A2.5.2.7.1.

The load carrying capacity of exterior beams shall not be less than the load carrying capacity of an interior beam, unless specifically approved by the Chief Bridge Engineer.

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From a structural viewpoint, large deflections in wood components cause fasteners to loosen and brittle materials, such as asphalt pavement, to crack and break. In addition, members that sag below a level plane present a poor appearance and can give the public a perception of structural inadequacy. Deflections from moving vehicle loads also produce vertical movement and vibrations that annoy motorists and alarm pedestrians, Ritter (1990).

Excessive deformation can cause premature deterioration of the wearing surface and affect the performance of fasteners, but limits on the latter have not yet been established.

The intent of the relative deflection criterion is to protect the wearing surface from debonding and fracturing due to excessive flexing of the deck. The restriction on relative rib displacement may be revised or removed when more data is available to formulate appropriate requirements as function of thickness and physical properties of the wearing surface employed.

C2.5.2.7.1

The following shall supplement A2.5.2.7.1.

The stiffness of the interior and exterior beams should be relatively equal.

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2.5.3 Constructibility

The following shall supplement A2.5.3.

An acceptable slab placement sequence shall be shown in the contract plans. Figure 2.5.3P-1 illustrates the format to be used. The actual sequence shall be determined from an erection analysis (see D6.10.3.2.5P) for the specific structure in question. For steel girder structures and prestressed beams made continuous for live load, see D6.10.3.2.5.1P and D5.14.1.4.12P, respectively, for additional requirements concerning slab placement sequence. The required curing strength (if any) of the concrete of a previous placement segment shall be designated as appropriate. Instead of specifying a curing strength of the concrete, a time delay (if any) between placements may be designated, as appropriate. A minimum waiting period of two curing days between positive moment region placements in immediately adjacent continuous spans and between adjacent positive moment placements in the same span shall be specified in the contract plans. A minimum waiting period between other placements need not be specified if analysis suggests that it is unnecessary.

COMMENTARY

C2.5.3

The following shall supplement AC2.5.3

The two-curing day waiting period between pours in adjacent continuous spans is for crack control. Studies have shown that longer waiting periods have no significant effect on cracking, primarily because shrinkage is the dominating factor in cracking. The two-curing day period between adjacent pours in the same span will provide enough strength gain to introduce composite action and will increase the stability of the girder over the length of the previous pour.

3	7	4	8	6	2	6	9	5	1	5	9	6	2	6	8	4	7	3
+M	-M	+M	-M	+M	-M	+M	-M	+M	-M	+M	-M	+M	-M	+M	-M	+M	-M	+M
○	○		○				△				△				○		○	○

Two days must elapse between Pours 1 and 2; 3 and 4; 5 and 6.

SLAB REPLACEMENT SEQUENCE

+M DENOTES POSITIVE MOMENT

-M DENOTES NEGATIVE MOMENT

NOTE: The Contractor may use an alternative slab placement sequence if the provisions in the DM-4, Section 2 (D2.5.3) are met.

Figure 2.5.3P-1 – Example of Slab Placement Sequence for Contract Plans

The Contractor may use an alternate slab placement sequence if the following provisions are met:

- The Contractor shall submit to the Department a revised slab placement sequence with support calculations and computer stress analysis. The revised slab placement sequence shall meet the requirements of which the original slab placement sequence were based on.
- The Department will review and approve calculations.
- The Contractor shall receive written approval prior to the use of the revised slab placement sequence and/or camber values.

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- All costs for the development and approval of the revised slab placement sequence and camber values shall be borne by the Contractor.
- The Department will be the sole judge of the acceptability of the revised slab placement sequence and camber values.

2.5.3.1P Falsework**C2.5.3.1P**

Composite beams shall be designed with no intermediate falsework during placing and curing of the concrete deck.

When falsework is used on a project, it shall be designed for the following items, but not limited to:

- vertical loads,
- horizontal loads,
- differential settlement forces,
- unbalanced temporary loadings (e.g., staged construction), and
- errant highway vehicles.

The following guidelines should be used for the approval of the falsework:

- Every bridge on a project should receive a separate falsework design analysis.
- In the event falsework is moved from one bridge to another, it should be thoroughly inspected for structural damage and plumbness to ensure that all members are in place and properly aligned and corrected.
- Ensure that the requirement of Publication 408, Section 105.02(c), “all drawings for load bearing falsework submissions are to be signed and sealed by a Professional Engineer, registered in the Commonwealth of Pennsylvania”, is fully enforced.
- During the falsework review, make sure that it is designed to handle vertical and horizontal loading and to contain enough redundancy to prevent a failure in the entire system. Vertical loading and differential settlement forces, and horizontal lateral and longitudinal forces should be taken into account. Unbalanced temporary loading caused by the placement sequence, should also be considered.
- If an unfortunate event occurs due to the failure of the falsework, preserve and document the in-place failure and assign investigation responsibilities to qualified impartial parties.
- If service load design is used, designers may increase the allowable basic unit stress by 25% for temporary falsework.

Refer to Appendix P for guidance on jacking and supporting the superstructure.

For purposes of these guidelines, temporary falsework is defined as falsework constructed for no more than one construction season.

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If the contractor or erector determine that temporary falsework is necessary for the erection of a curved or skewed steel structure, the following guidelines should be used for determining the placement locations:

- When temporary falsework is needed for a span, it shall be placed at locations to reduce splice rotations and girder vertical deflections.

The stability of the structure supported by temporary falsework shall be evaluated.

2.5.3.2P Girder Erection Sequence

The following guidelines should be used for the erection sequence of horizontally curved steel I-girder structures:

- If adequate crane capacity is available, a paired girder erection approach is preferred.
- When the radius of the curved structure is less than 300 feet, it is recommended that the girders be placed from the inner radius to the outer radius.
- An analysis shall be performed to ensure that the structure is stable for all stages of erection and that supports necessary to maintain stability have been provided.

The following guidelines should be used for the erection sequence of skewed steel I-girder structures:

- An analysis shall be performed to ensure that the structure is stable for all stages of erection and that supports necessary to maintain stability have been provided.

2.6 HYDROLOGY AND HYDRAULICS**2.6.4 Hydraulic Analysis****2.6.4.4 Bridge Foundations**

COMMENTARY

The following information is based on Penn State Research Report titled Guideline for Analyzing Curved and Skewed Bridges and Designing them for Construction, August 15, 2010.

While using no temporary falsework is desirable from a cost-effectiveness perspective, should the designer and/or the contractor/erector determine that falsework is needed to ensure that a curved or skewed steel structure is constructible, it should initially be placed near the splice locations. When girder vertical deflections are a concern, additional temporary supports should be placed as close as possible to the location of maximum vertical deflection of the span (approximately 0.4 L from an abutment, where L is the span length, for end spans and 0.5 L for intermediate spans) to reduce girder deflections. For the end spans, when adding multiple temporary supports is not feasible, placing one support at approximately 0.75 L from the abutment is suggested.

C2.5.3.2P

Paired girder erection, as opposed to single girder erection, requires fewer temporary supports for the erected segments during all stages of erection. However, for structures with an odd number of girders, at least one girder line must be erected individually.

Girder erection from the inner radius to the outer radius, when compared to the opposite direction, can result in slightly smaller deformations for the girders for all stages of erection which, in turn, means that structure is more constructible. This effect is more pronounced in severely curved structures (i.e. radius less than 300 feet).

Stability, which refers to the prevention of excessive deformations and the possibility of buckling, of partial and completed girders at various stages of erection, is the responsibility of the contractor as specified in Publication 408, Section 1050.3(c).

For erection of straight skewed structures, paired girder erection does require a smaller number of temporary supports; however, paired girder erection offers no other substantial benefits over the single girder erection approach with respect to deflections.

The information in this Article is based on Penn State Research Report titled Guideline for Analyzing Curved and Skewed Bridges and Designing them for Construction, August 15, 2010.

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2.6.4.4.2 Bridge Scour

C2.6.4.4.2

The following shall replace the second bullet point of A2.6.4.4.2.

The following shall supplement AC2.6.4.4.2.

- For the check flood for scour, the stability of the bridge foundations shall be investigated for scour conditions resulting from a superflood, as defined in PP7.2.3. Excess reserve beyond that required for stability under this condition is not necessary. The Service I limit state shall apply.

Investigate the stability due to the maximum scour at the Service I limit state to ensure the bridge can remain operational until the extent of damage can be evaluated and repaired.

The following shall supplement A2.6.4.4.2.

For additional guidance on bridge scour, footing placement criteria, and scour countermeasures, see Policies and Procedures, Chapter 7.

2.6.6 Roadway Drainage

2.6.6.1 General

The following shall supplement A2.6.6.1.

Dimensions for the deck cross slopes shall be shown in the same manner as indicated on the roadway plans (e.g., 2%).

The water table cross slope on bridge decks which are not superelevated shall be sloped toward the curb or median. The rate of slope shall be 4% for water table widths of 6 ft. or less, and 3% for water table widths over 6 ft.. In rare cases the water table may be designed to be a Pedestrian Accessible Route (PAR) which requires a 2% maximum cross slope. See Design Manual, Part 2, Chapter 6, Section 6.5.B.4. On superelevated decks, the water table on the high side shall be as specified in Design Manual, Part 2. On the low side, the water table shall slope in the same direction and magnitude as the adjacent lane, but not less than 4% for water table widths of 6 ft. or less, nor less than 3% for water table widths over 6 ft.. On a superelevated bridge with a paved median, adjustment of the grades of adjacent roadways may be required to equalize the height of the divider or median barrier.

2.7 BRIDGE SECURITY

2.7.1 General

The following shall replace A2.7.1.

For the purpose of this section, bridges deemed important shall include; (1) new singular bridges with a total replacement value exceeding \$100 million or (2) new or existing bridges identified as critical by the Department's Emergency Transportation Operations Section.

A risk management approach shall be utilized to assess structural vulnerability and countermeasures. A threat based component level analysis shall be conducted considering a full range of threats. FHWA's workshop methodology or other BDTD approved methodology shall be used.

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COMMENTARY

The results of such processes and included design/mitigation criteria and features shall be considered sensitive project information that is protected by restricted access. Contract documents shall not include reference to any security standard, design capability, or other information that might provide knowledge of bridge resistance.

For all bridges, restrict access to doors/hatches by using locks or by making inaccessible except by special mobile equipment, such as snooper, man lift, etc. In cellular structures, avoid vent hole diameters larger than 2 inches when holes are in easy reach. Avoid nooks and areas that allow for concealed access and create a confined pressure effect. If these details are unavoidable consideration should be given to barring access.

2.9P DESIGN DRAWINGS**2.9.1P Moment and Shear Envelope Diagrams**

For simple span bridges, the contract plans shall contain a table of the following items:

- (1) Maximum non-composite dead load moment
- (2) Maximum composite dead load moment (including future wearing surface)
- (3) Maximum live load plus impact moment for PHL-93 and P-82 loading conditions
- (4) Maximum non-composite dead load shear
- (5) Maximum composite dead load shear (including future wearing surface)
- (6) Maximum live load plus impact shear for PHL-93 and P-82 loading conditions
- (7) Composite and non-composite section properties at resisting sections

For multiple span continuous bridges, as a minimum, the contract plans shall contain a diagrammatic presentation of the following on a per-girder basis:

- (1) Non-composite dead load moment diagram
- (2) Composite dead load moment diagram (including future wearing surface)
- (3) Separate positive and negative live load plus impact moment envelope for PHL-93 and P-82 loading conditions
- (4) Summation of (1) and (2)
- (5) Non-composite dead load shear diagram
- (6) Composite dead load shear diagram (including future wearing surface)

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- (7) Separate positive and negative live load plus impact shear envelope for PHL-93 and P-82 loading conditions
- (8) Summation of (5) and (6)
- (9) Composite and non-composite section properties at the resisting sections

The data to construct this presentation shall not have load factors applied.

Also, a table of reactions shall be provided for total dead load, and positive and negative live load, plus impact without load factors applied.

2.9.2P Major, Complex and Unusual Bridges

2.9.2.1P Load Data Sheet

For the new design of major, complex and unusual bridges, additional information shall be included on the design drawings for typical common components such as bearings, floorbeams, and stringers. For these items as a minimum, the contractor plans shall contain a tabular presentation of the following:

- Bearings
 - (a) Vertical Force
 - 1. Total Dead Load
 - 2. Live Load plus Impact
 - 3. Summation of 1. and 2.
 - (b) Transverse Force Wind Load
 - (c) Longitudinal Force
 - 1. Wind Load
 - 2. Traction Load
 - 3. Friction Load
- Truss Members
 - (a) Axial Force
 - 1. Total Dead Load
 - 2. Live Load plus Impact
 - 3. Summation of 1. and 2.
 - (b) Bending Moment
 - 1. Total Dead Load
 - 2. Live Load plus Impact
 - 3. Summation of 1. and 2.
 - (c) Section Properties
 - 1. Gross Area
 - 2. Net Area
 - 3. Section Modulus
- Floorbeam

Provide the same type of information as required in D2.9.1P for girders.

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- Stringers
Provide the same type of information as required in D2.9.1P for girders.

The data for this tabular presentation shall not have load factors applied.

2.9.3P Skewed Steel Bridges

For the new design of skewed steel bridges, additional information shall be included on the design drawings for top flange lateral displacement as given in D6.7.2. For these items as a minimum, the contractor plans shall contain a schematic and tabular presentation of the anticipated lateral displacements.

2.10P CONSTRUCTION LOADS**C2.10P**

Construction loads must be considered when developing a preservation or rehabilitation strategy as well as for new bridges constructed under phased operations which may simultaneously support live traffic and construction loads. The bridge contract documents (structure plans or applicable design-build special provision) shall include the applicable General Note given below in D2.10.1P. Additionally, the bridge contract documents for Bridge Painting projects and/or Deck Milling/Overlay projects are also to include the Construction Load Capacity given in D2.10.2.1P and D2.10.2.2P, respectively. The bridge contract documents for projects requiring a crane located on the bridge shall include the crane evaluation, analysis, and plan information given in D2.10.2.3P. The consideration of construction loads in permit evaluation is given in D2.10.4P. Structural analysis calculations and capacities for construction loads on bridges shall be based on actual field conditions.

At the District Bridge Engineer's request, separate load limits may be specified for different components or stages if the controlling rating factor differs significantly between the components and/or stages.

Field Conditions" means the member or element condition (such as section loss, dead loads, etc.) must be considered – much like the Department's bridge load ratings must consider any detrimental conditions noted during routine NBIS inspections.

2.10.1P General Note**C2.10.1P**

Add the applicable General Note below to the structure plans or to the applicable design-build special provision. For Item 2, the designer shall determine and indicate the reduced construction loading limits based on the rating factor controlling the weight restriction.

1. "Bridge is not weight restricted. See Publication 408 Section 105.17 for construction loading limits."
2. "Bridge is posted for a weight restriction of ____ Tons. Construction loadings limits have been reduced in accordance with Publication 408 Section 105.17."

The temporary barrier weight does not count against the construction loading limits given by items 1 and 2. However, the temporary barrier weight is to be included with the factored dead load in determining the Construction Load Capacities given by D2.10.2P, as appropriate.

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The reduced construction loading limits are:

- a. Individual material stockpile (including but not limited to pallets of products, reinforcement bar bundles, aggregate piles) - Limited to one individual stockpile with a Maximum weight of ____ pound/square foot and a Maximum size of 100 square feet.
- b. Multiple material stockpiles - Maximum weight of ____pound/square foot with a Maximum total stockpile area of 1000 square feet provided loading limit for an individual material stockpile (above) is not exceeded.
- c. Combinations of material stock piles, vehicles, other materials, and equipment are limited to a Maximum total weight of ____ pound per span in any work zone width less than 24 feet and limited to a Maximum total weight of ____ pound per span for work zones ≥ 24 feet wide provided loading limits for an individual material stockpile and multiple material stockpiles (above) are not exceeded.”

2.10.2P Project Specific Requirements

COMMENTARY

C2.10.2P

The Project Specific Requirements given by D2.10.2P relate to simple projects involving Bridge Painting or Deck Milling / Overlays. The following observations were considered in addressing these concerns:

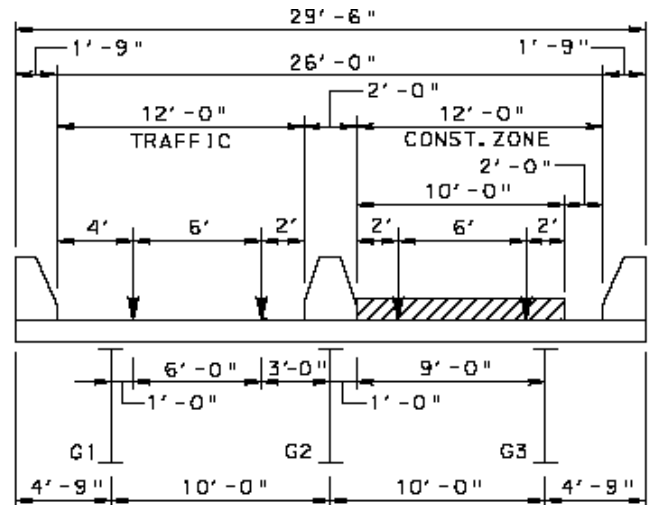
The development of the project specific requirements recognized the need for a simple but reasonable method of determining the possible loading effects from an endless number construction loading arrangements combined with the effects from the many different design/rating vehicle configurations. The approach herein is based upon the simple observation that a live load distribution factor can represent the combined effects of multiple “lanes” of loading on a given member. The actual rating vehicle type is not indicated because their effect, combined with the construction loads, is considered to be accounted for via the member distribution factor, rather than actual vehicle weight. Strictly speaking, this implies that the longitudinal and transverse configuration of the construction loading matches that of the rating vehicle under consideration. However, rather than taking that the construction loading matches the rating vehicle(s), consider that the vast majority of the construction loading arrangements are enveloped within the effects produced by the various rating vehicles.

The construction loads distribution factor is determined in the same way as for a design or rating vehicle. Here, the simplifying assumption is that the transverse arrangement of the rating vehicle (which may be a “truck” or a “lane”) adequately envelopes those of the construction loads. This is considered to be a reasonable assumption given that 1) the

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standard AASHTO 12 foot “lane” consists of a 10 foot loaded zone which may laterally float up to two feet within the 12 foot lane, and 2), since the 10 foot loaded zone consist of either a truck (wheel loads) or a lane load (distributed over the entire 10 foot width). See Figure C2.10.2P-1.

**EXAMPLE DISTRIBUTION CALCULATION:**

$$G2: DF_{\text{TRAFFIC}} = \frac{1.0}{10.0} + \frac{7.0}{10.0} = 0.8 \text{ WHEELS} = 0.4 \text{ LANES}$$

$$G2: DF_{\text{CONST. ZONE}} = \text{FOR THIS BRIDGE/CASE, SAME AS } DF_{\text{TRAFFIC}} = 0.4 \text{ LANES}$$

THUS, THE LIVE LOAD DISTRIBUTION FACTOR USED TO CALCULATE THE CONSTRUCTION LOAD CAPACITY IS 1.6 WHEELS OR 0.8 LANES.

Figure C2.10.2P-1

Note that the Construction Load Capacity Equation may give elevated results for shear at end supports. Designers need to consider the mechanisms driving the actual capacity. Designers should also consider other structural components such as bearings, pier caps, etc. and reduce the capacity envelopes appropriately.

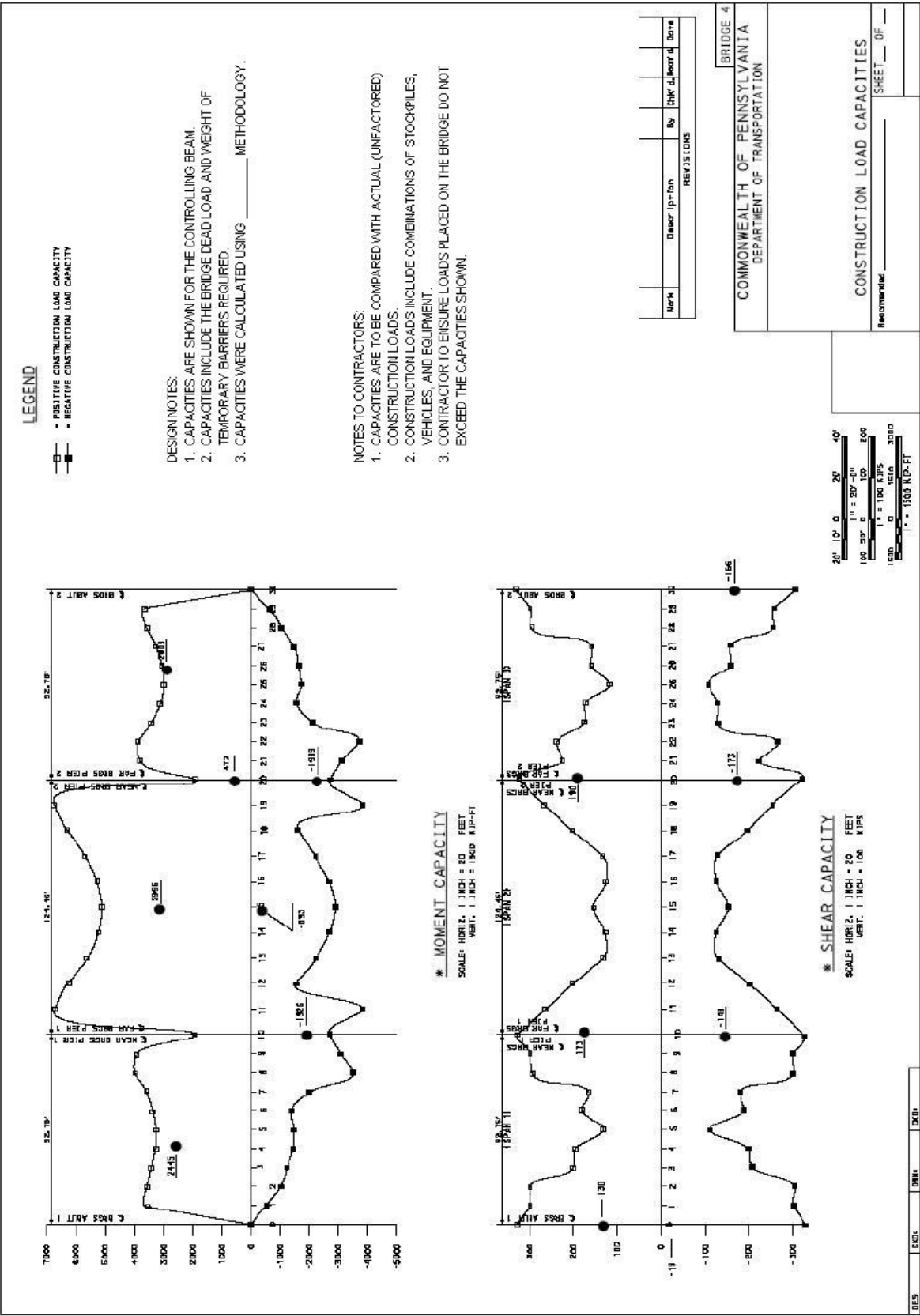


Figure 2.10.2P-1

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2.10.2.1P Bridge Painting Projects

The Designer shall perform the following:

1. Determine Construction Load Moment and Shear Capacities for the governing interior and exterior member(s) as applicable. The Construction Load Capacities may be based on:
 - a. Construction Load (Moment or Shear) Capacity
= (Operating Capacity – Factored Dead Load)/(Impact × Live Load Distribution Factor × Load Factor)
2. Provide a drawing or table showing the Construction Load Moment and Shear Capacity envelopes for the governing interior and exterior member(s). See Figure 2.10.2P-1 for an example drawing showing the Construction Load Capacities. The temporary barrier weight shall be included with the Factored Dead Load.
3. The Construction Load Capacities shall be reduced to account for the portion of the capacity supporting vehicular live load in the open lane(s) where the superstructure is comprised of three (3) or less main member (i.e. beam, girder, truss, etc.) lines. Vehicular live load effect shall be based on lever rule distribution.
4. For two-line bridge superstructures, provide Construction Load Capacity drawings or tables for each governing load carrying member: main members (girder or truss), floorbeam members, and stringer members.
5. A general note indicating that the painting operations must be staged off the bridge and that the construction loadings are restricted to those given in D2.10.1P may be used in lieu of D2.10.2.1P Items 1 through 4 (NOTE: This option has the potential to increase bid prices.)

2.10.2.2P Deck Milling / Overlay Projects

The Designer shall perform the following:

1. Determine Construction Load Moment and Shear Capacities for the governing interior and exterior member(s) as applicable. The Construction Load Capacities may be based on:
 - a. Construction Load (Moment or Shear) Capacity
= (Operating Capacity – Factored Dead Load)/(Impact × Live Load Distribution Factor × Load Factor)
2. Provide a drawing or table showing the Construction

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Load Moment and Shear Capacity envelopes for the governing interior and exterior member(s). See Figure 2.10.2P-1 for an example drawing showing the Construction Load Capacities. The temporary barrier weight shall be included with the Factored Dead Load.

3. The Construction Load Capacities shall be reduced to account for the portion of the capacity supporting vehicular live load in the open lane(s) where the superstructure is comprised of three (3) or less main member (i.e. beam, girder, truss, etc.) lines. Vehicular live load effect shall be based on lever rule distribution.
4. For two-line bridge superstructures, provide Construction Load Capacity drawings or tables for each governing load carrying member: main members (girder or truss), floorbeam members, and stringer members.
5. The District may choose to have the designer perform constructability analyses instead of including the Construction Load Capacity envelopes for mill and overlay projects. Prior to selecting this option, the District must submit the proposed construction loading configurations to BDTD for approval at least 6 months in advance of final plan submittal. The proposed construction loading configurations shall include the actual milling machines and associated equipment typically utilized on District projects. For projects implementing this option, the designer must specify in the General Notes or applicable design-build special provision the actual milling machines and loading configurations that were analyzed. The designer may only use approved construction loading configurations in the development of the final plans.

2.10.2.3P Cranes on Bridges

When the project conditions require that a crane be located on the bridge, the designer shall perform the following:

1. Select a proper crane size, boom length, and pick radius for the required lift(s).
2. Analyze the structure for crane location, size, operating weight and pick weight.
3. Provide the following on the plans: crane location, size, operating weight and pick weight.

C2.10.2.3P

The requirements given for “Cranes on Bridges” is applicable only to projects which specifically (contractually) require the contractor to locate a crane on the bridge: i.e., projects where no temporary or permanent Right-Of-Way is available or obtained, or projects where the site constraints preclude the contractor from locating a crane below, next to, or near the bridge. For these types of project, the designer is obligated and required to assure that the project is constructible. The designer’s assumptions / approach must be reasonable and show that at least one method, which should be verified with an independent constructability analysis, is available.

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2.10.2.4P Construction Loads and Permit Evaluations

In the development of the bridge design and/or approval of construction loads, the designer shall consider the effects of construction loads in conjunction with permit load evaluation. The District shall adjust the APRAS permit evaluation load rating data and conditions for the bridge to ensure the combined effects of construction loads and permit vehicles do not overstress a bridge member or component. One approach that ensures the combined effects do not cause overstress is for each analysis to consider all lanes loaded. The construction load analysis would take the maximum construction load proposed in any one lane, and place it in each design lane. The separate permit load evaluation would place the permit vehicle in each design lane. An exception to this may occur when exterior girders of multi-girder bridges are analyzed by the Load and Resistance Factor method, in which case maximum force can occur from less than all lanes loaded.

2.11P REFERENCES

See A2.8 for references.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 3 – LOADS AND LOAD FACTORS

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3.1 SCOPE

C3.1

The following shall supplement the second paragraph of AC3.1.

Before any test results are used in the design of a structure, the tests and the test results must be approved by the Chief Bridge Engineer.

3.3 NOTATION

3.3.1 General

The following shall supplement A3.3.1

- B = Vertical element width (ft) (A3.11.5.6)
- C = Pressure coefficient for loads applied on a subgrade (dim) (D3.6.1.5.2P)
- D = Depth of embedment of discrete and continuous vertical wall elements (ft) (A3.11.5.6); width of uniformly loaded area (ft) (D3.6.1.5.2P)
- F = Fictitious force applied at bottom of embedded continuous vertical wall element to provide horizontal force equilibrium for simplified earth pressure distributions (kip/ft) (A3.11.5.6)
- I_f = Impact factor (dim) (D3.6.1.5.2P)
- k_p = Passive coefficient of lateral earth pressure (dim) (A3.11.5.6)
- P = Spacing between vertical wall elements (c/c) (ft) (A3.11.5.6)
- M = Length of uniformly loaded area (ft) (D3.6.1.5.2P)
- m = constant used in calculating active earth pressure coefficient in certain conditions (dim) (D3.11.5.7.2b)
- P_a = Active resistance per vertical wall element (kips)
- P_a = Active earth pressure per unit length of wall (kip/ft) (A3.11.5.6)
- P_o = Intensity of the distributed load at the bottom of the railroad ties (ksi) (D3.6.1.5.2P)
- P_p = Passive resistance per vertical wall element (kips)
- P_p = Passive earth pressure per unit length of wall (kip/ft) (A3.11.5.6)
- S_m = Shear strength of rock mass (ksf) (A3.11.5.6)
- S_u = Undrained shear strength of cohesive soil (ksf) (A3.11.5.6)
- W_1 = Live load on structure from railroad loading (kip/ft) (D3.6.1.5.2P)
- β = Ground surface slope behind wall {+ for slope up from wall; - for slope down from wall} (degrees) (A3.11.5.6)
- β' = Ground surface slope in front of wall {+ for slope up from wall; - for slope down from wall} (degrees) (A3.11.5.6)

3.4 LOAD FACTORS AND COMBINATIONS

3.4.1 Load Factors and Load Combinations

C3.4.1

The following shall replace Strength II description in A3.4.1.

The following shall replace Strength II commentary in AC3.4.1.

- Strength II - Load combination relating to the Design Permit Load (P-82) use of the bridge. This load combination only applies to the superstructure, except for pier caps which support a superstructure with a span length greater than 65 ft.. Bearings, (including uplift check), substructure and foundation need not be designed for this load combination. For design, the distribution factors given in D4.6.2.2 and A4.6.2.2 shall be used.

In design, the use of distribution factors in D4.6.2.2 and A4.6.2.2 represents that the P-82 is in all design lanes.

The method for rating takes into account that the P-82 is in one lane and the other lanes are occupied by the vehicular live load.

For bridges longer than the permit vehicle, the presence of the design lane load, preceding and following the permit load in its lane was studied by the Department. The study showed that the P-82 with the interrupted lane load only controls for moments in a small range of spans and is only maximum of 2% above the PHL-93 loading. For shear, the maximum difference between the PHL-93 and P-82 with lane load was 7.5% with P-82 and lane load being greater

For the rating of existing bridges with Strength II

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criteria, the following equation may be used to determine Strength II live load moments and shear:

$$FR_T = FR_{P-82} \left(\frac{g_1}{Z} \right) + FR_{PHL-93} \left(g - \frac{g_1}{Z} \right) \quad (3.4.1-3P)$$

where:

FR_T = total force response, moment or shear

FR_{P-82} = P-82 force response, moment or shear

FR_{PHL-93} = PHL-93 force response, moment or shear

g_1 = single lane distribution factor, moment or shear

g = multi-lane distribution factor, moment or shear

Z = a factor taken as 1.20 where the lever rule was not utilized, and 1.0 where the lever rule was used for a single lane live load distribution factor

FR_T need not be taken greater than $FR_{P-82(g)}$.

The following shall supplement A3.4.1.

- STRENGTH IP - Load combination relating to the pedestrian load and a reduced vehicular live load.
- STRENGTH VI - Load combination relating to the design of piers which includes ice and wind load acting together.
- EXTREME EVENT III - Load combination relating to the failure of one element of a component without the failure of the component.
- EXTREME EVENT IV - Load combination relating to the failure of one component without the collapse of the structure.

The conditions for which Extreme Event III and IV are to be investigated are given in D1.3.4.

COMMENTARY

than PHL-93. The Department concluded that this difference was acceptable because the study considered all the lanes loaded with the P-82. Therefore, the P-82 loading need not be considered with a partial lane load.

The following shall supplement AC3.4.1.

Extreme Events III and IV are uncalibrated load combinations. They are intended to force consideration of the safety of damaged structures.

For this extreme event, a 3-D analysis is required. The objective of this analysis is survival of the bridge (i.e., the bridge may have large permanent deflections, but it has not collapsed).

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Table 3.4.1P-1 - Additional PennDOT Load Combinations and Load Factors

Load Combination	DC DD DW EH EV ES PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
Limit State													
STRENGTH IP	γ_p	*	-	-	-	-	-	-	-	-	-	-	-
STRENGTH VI	γ_p	-	1.25	1.25	-	-	-	-	-	-	1.25	-	-
EXTREME EVENT III	γ_p	γ'_{LL}	-	-	-	-	-	-	-	-	-	-	-
EXTREME EVENT IV	γ'_p	γ'_{LL}	-	-	-	-	-	-	-	-	-	-	-
* $\gamma_{LL} = 1.35$, $\gamma_{PL} = 1.75$													

Table 3.4.1P-2 - Load Factor for Live Load for Extreme III and IV, γ'_{LL}

Case	III	IV
	γ'_{LL}	γ'_{LL}
PHL-93 Loading – all applicable lanes	1.30	1.15
Permit load in governing lane with PHL-93 in other applicable lanes	1.10	1.05

Table 3.4.1P-3 - Load Factors for Permanent Loads for Extreme Event IV, γ'_p

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.05	0.95
DW: Wearing Surfaces and Utilities	1.05	0.90

Unless otherwise specified, interaction of force effects shall be accounted for by selecting load factors which maximize and minimize each of the force effects one at a time with the same load factors used to compute the associated force effect.

As an example for a design which involves the interaction of moment and axial force, the following four design cases would be investigated:

- select the load factors which maximize moment and use these load factors in determining axial force
- select the load factors which minimize moment and use these load factors in determining axial force
- select the load factors which maximize axial force and use these load factors in determining moment

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The following shall replace the ninth paragraph of A3.4.1 relating to γ_{TG} .

For the application of temperature gradient see D3.12.3.

The following shall replace the tenth paragraph of A3.4.1 relating to γ_{SE} .

The load factor for settlement γ_{SE} shall be determined on a project-specific basis.

The following shall replace the thirteenth paragraph of A3.4.1 relating to γ_{EQ} .

The load factor γ_{EQ} for live load for the Extreme Event-I limit state shall be taken as 0.0.

The following shall supplement A3.4.1 for the design of box culverts.

Lateral earth pressures for box culverts shall be computed using the equivalent fluid method given in A3.11.5.5 and D3.11.5.5, and appropriate load factors, EH, as given in Table A3.4.1-2, for horizontal earth pressures.

To maximize the load effect, the maximum at-rest load factor shall be used with the maximum equivalent fluid weight from Table D3.11.5.5-2P, and the minimum at-rest load factor shall be used with the minimum equivalent fluid weight. In addition, a 50% reduction in both maximum and minimum unfactored lateral earth pressures, EH and ES, shall be considered for determining the maximum positive moment in the top slab of the culvert, as specified in A3.11.7.

3.4.1.1P Load Factors and Combinations for Typical PennDOT Bridge Components

Tables 3.4.1.1P-1 through 3.4.1.1P-6 provide the load factors with the corresponding limit state condition for the following typical PennDOT bridge components:

COMMENTARY

- select the load factors which minimize axial force and use these load factors in determining moment.

Due to the nature of force interaction, the absolute worst case may not necessarily be that for which one of the force effects is maximized, but an intermediate case. However, the difference between the absolute worst case and the design cases presented here are believed to be within the tolerance of the design process. Therefore, as a reasonable interpretation of the specification, maximum and minimum force effects taken in conjunction with associated force effects for interaction are to be considered. If the Engineer believes that an intermediate case will govern to an appreciable degree, the Engineer shall notify the Chief Bridge Engineer. Then, the Chief Bridge Engineer will determine if intermediate cases shall be investigated.

For MSE wall designs, D11.10.5.2 and D11.10.6.2 state when to apply maximum and minimum EH and EV.

The Department is currently using $\gamma_{EQ} = 0.0$ in accordance with numerous past years of AASHTO practice. We will continue to use $\gamma_{EQ} = 0.0$ until further work is completed justifying a different value.

The following shall supplement AC3.4.1 for the design of box culverts.

Rigid frame action of box culvert structures is assumed to result in relatively small movements as compared to that of a retaining wall or abutment-type structure, thus, an at-rest condition is assumed.

C3.4.1.1P

The design live load vehicle in the fatigue load combination designated as HS20-30 refers to an HS20 truck with a fixed 30 ft. rear axle spacing.

SPECIFICATIONS**COMMENTARY**

- steel girders (Table 3.4.1.1P-1)
- prestressed girders (Table 3.4.1.1P-2)
- abutment/retaining walls (Table 3.4.1.1P-3)
- box culverts (Table 3.4.1.1P-4)
- steel floorbeams (Table 3.4.1.1P-5)
- steel trusses (Table 3.4.1.1P-6)

Tables 3.4.1.1P-1 through 3.4.1.1P-6 also include information for rating these components. (Ratings are not typically done for abutment/retaining walls.)

For the design of anchored walls, see Standard Drawing BD-626M for the load factors and corresponding limit states.

Table 3.4.1.1P-1 - Load Factors and Live Load Vehicles for Steel Girders

	Load Combination													
	STR I	STR IP ⁸	STR IA ⁶	STR II	STR III	STR IV ¹	STR V	SERV II	SERV IIA ⁶	SERV IIB	Fatigue I ² (infinite)	Fatigue II ² (finite)	DEFL ²	CONST/ UNCURED SLAB ⁹
γ_{DC} ³	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.5	1.25, 0.90	1.00	1.00	1.00	---	---	---	1.25
γ_{DW} ⁴	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.00	1.00	1.00	---	---	---	1.50, 0.65
γ_{LL}	1.75	1.35	1.35	1.35	---	---	1.35	1.30	1.00	1.00	1.50	0.75	1.00	1.50
γ_{PL}	---	1.75	---	---	---	---	---	---	---	---	---	---	---	---
γ_{WS}	---	---	---	---	1.40	---	0.40	---	---	---	---	---	---	1.25
Design LL Veh. ⁷	PHL-93	PHL-93	PHL-93	Permit (P-82)	---	---	PHL-93	PHL-93	PHL-93	Permit (P-82)	HS20- 30	HS20- 30	PennDOT Defl. Trk.	User Def.
Rating Veh.	Rating Applicability: I = Inventory, O = Operating													
PHL-93	I	I	O	---	---	---	---	I	O	---	---	---	---	---
P-82	---	---	---	O	---	---	---	---	---	O	---	---	---	---
ML-80	I	I	---	O	---	---	---	I	O	---	---	---	---	---
TK527	I	I	---	O	---	---	---	I	O	---	---	---	---	---
HS20	I	I	---	O	---	---	---	I	O	---	---	---	---	---
H20	I	I	---	O	---	---	---	I	O	---	---	---	---	---
Spec. Veh.	I	I	---	O	---	---	---	I	O	---	---	---	---	---

Table 3.4.1.1P-1 - Load Factors and Live Load Vehicles for Steel Girders (Continued)

Notes:

¹Applicable when DL/LL ratio exceeds 7.0²A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified³DC load factor also used for barrier loads⁴DW load factor also used for utility loads⁵All loads applied to non-composite section for non-composite girders (Live loads are applied to the n section for steel)⁶Load combination for rating only⁷This row lists the typical design vehicle to be used for each load combination⁸The reduced load factor for LL with PL (see D3.4.1)⁹Design live load N/A for uncured slab check

Permanent Loads for Girder Programs		Section Properties ⁵	
Load	Steel		Steel
DC1	γ_{GR}	Girder	nc
	γ_{SLAB}	Slab	nc
	γ_{SLAB}	Haunch	nc
DC2	γ_{DC2}	Barrier	3n
DW	γ_{FWS}	FWS	3n

Table 3.4.1.1P-2 - Load Factors and Live Load Vehicles for Prestressed Concrete Girders

	Load Combination											
	STR I	STR IP ⁸	STR IA ⁶	STR II	SERV I (P/S compr. chk.)		SERV III (P/S tension chk.)		SERV IIIA (M _r @ 0.9 f _y chk.)	SERV IIIB (PennDOT- cracking chk.)	Fatigue I ¹ (infinite)	DEFL ¹
					w/o PL	with PL	w/o PL	with PL				
γ _{DC} ²	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.00	1.00	1.00	1.00	1.00	1.00	---	---
γ _{DW} ³	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.00	1.00	1.00	1.00	1.00	1.00	---	---
γ _{LL}	1.75	1.35	1.35	1.35	1.00	0.80	0.80 ⁴	0.65 ⁹	1.00	1.00	1.50	1.00
γ _{PL}	---	1.75	---	---	---	1.00	---	1.00	---	---	---	---
γ _{CR,SH}	0.5	0.5	0.5	0.5	1.00	1.00	1.00	1.00	---	---	---	---
Design LL Veh. ⁷	PHL-93	PHL-93	PHL-93	Permit (P-82)	PHL-93		PHL-93		Controlling PHL-93 or P-82	Controlling PHL-93 or P-82	HS20-30	PennDOT Defl. Trk.
Rating Vehicle	Rating Applicability: I = Inventory, O = Operating											
PHL-93	I	I	O	---	I		I		O	---	---	---
P-82	---	---	---	O	---		---		O	---	---	---
ML-80	I	I	---	O	I		I		O	---	---	---
TK527	I	I	---	O	I		I		O	---	---	---
HS20	I	I	---	O	I		I		O	---	---	---
H20	I	I	---	O	I		I		O	---	---	---
Spec. Veh.	I	I	---	O	I		I		O	---	---	---

Table 3.4.1.1P-2 - Load Factors and Live Load Vehicles for Prestressed Concrete Girders (Continued)

Notes:

¹A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified²DC load factor also used for barrier loads³DW load factor also used for utility loads⁴For rating vehicles, the live load for Service III is to be taken as 1.0 ($\gamma = 0.80$ for PHL-93 only)⁵All loads applied to non-composite section for non-composite girders

(Live loads are applied to the n section for P/S composite girders. For P/S, live load stresses can be based on transformed strands)

⁶Load combination for rating only⁷This row lists the typical design vehicle to be used for each load combination⁸The reduced load factor for LL with PL (see D3.4.1)⁹For rating vehicles (other than PHL-93), the live load factor for Service III is to be taken as 0.80 for the pedestrian load case

Permanent Loads for Girder Programs		Section Properties ⁵	
Load		P/S	P/S
DC1	γ_{GIR}	Girder	nc
	γ_{SLAB}	Slab	nc
	γ_{SLAB}	Haunch	nc
	γ_{ID}	Int. Dia.	nc
	γ_{ED}	Ext. Dia.	nc
DC2	γ_{DC2}	Barrier	n
DW	γ_{FWS}	FWS	n

Table 3.4.1.1P-3 - Load Factors and Live Load Vehicles for Abutment/Retaining Walls

	Load Combination									
	SERV I	STR I	STR IP	STR II	STR III	STR V	EXTREME I ²	EXTREME II ³	Min. γ for Const. Case (Strength) ⁵	γ for consolidation/secondary settlement
γ_{DC}	1.00	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.00
γ_{DW}	1.00	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	0.00	---	1.00
γ_{EV}	1.00	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}	1.00
γ_{EH}	1.00	γ_{EH}	γ_{EH}	γ_{EH}	γ_{EH}	γ_{EH}	0.00	γ_{EH}	γ_{EH}	1.00
γ_{ES}^4	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.00
γ_{LS}^4	1.00	1.75	1.35	1.35	0.00	1.35	γ_{EQ}	0.50	1.50	0.00
γ_{LLIM}^1	1.00, 0	1.75, 0	1.35, 0	1.35, 0	0.00	1.35, 0	$\gamma_{EQ}, 0$	0.00	---	0.00
γ_{PL}	0.00	0.00	1.75, 0.00	0.00	0.00	0.00	0.00	0.00	---	0.00
γ_{WS}	0.3	0.00	0.00	0.00	1.40	0.40	0.00	0.00	---	0.00
γ_{WL}	1.0	0.00	0.00	0.00	0.00	1.0	0.00	0.00	---	0.00
γ_{WA}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
γ_{BR}	1.0	1.75	1.35	1.35	0.00	1.35	γ_{EQ}	0.00	---	0.00
γ_{CE}	1.0	1.75	1.35	1.35	0.00	1.35	γ_{EQ}	0.00	---	0.00
γ_{FR}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	---	0.00
γ_{TU}	1.0	0.5	0.5	0.5	0.5	0.5	0.00	0.00	---	0.00
γ_{EQ}	0.00	0.00	0.00	0.00	0.00	0.00	1.0	0.00	---	0.00
γ_{CT}	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.0	---	0.00
Design LL Vehicle	PHL-93	PHL-93	PHL-93	P-82	---	---	PHL-93	---	---	—

Table 3.4.1.1P-3 - Load Factors and Live Load Vehicles for Abutment/Retaining Walls (Continued)

Notes:

¹For a negative reaction on an abutment (uplift), use the maximum load factor²For the seismic load case (refer to Appendix A10), EH loads (normal lateral earth pressure) replaced by EQ soil loads. γ_{EQ} for live loads = 0.0.³Parapet collision force, CT.⁴All lateral loads and their vertical components are maximized.⁵For evaluation of the temporary construction stages using the Strength Limit states (see D11.6.1.2), use the greater of the γ noted under Construction Case column or under the given Strength Limit State column.

	Abutment/Retaining Wall Earth Load Factors	
	Maximum	Minimum
γ_{EV}	1.35	1.00
γ_{EH}	1.50	⁽⁴⁾

Table 3.4.1.1P-4 - Load Factors and Live Load Vehicles for Box Culverts

	Load Combination				
	SERV I	STR I	STR IA	STR II	Min. γ for Const. Case (Strength) ⁴
γ_{DC}	1.00	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90
γ_{DW}	1.00	1.50, 0.65	1.50, 0.65	1.50, 0.65	---
γ_{EV}	1.00	γ_{EV}	γ_{EV}	γ_{EV}	γ_{EV}
γ_{EH}	1.00	γ_{EH}	γ_{EH}	γ_{EH}	γ_{EH}
γ_{ES}^3	1.00	1.50, 0.75	1.50, 0.75	1.50, 0.75	1.50, 0.75
γ_{LS}	1.00, 0	1.75, 0	1.35, 0	1.35, 0	1.50, 0
γ_{LLIM}	1.00, 0	1.75, 0	1.35, 0	1.35, 0	---
Design LL Vehicle	PHL-93	PHL-93	PHL-93	P-82	---
Rating Vehicle ²	Rating Applicability: I = Inventory, O = Operating				
PHL-93	---	I	O	---	---
P-82	---	---	---	O	---
ML-80	---	I	---	O	---
TK527	---	I	---	O	---
HS20	---	I	---	O	---
H20	---	I	---	O	---
Spec. Veh.	---	I	---	O	---

Notes:

¹Fatigue load factor should be factored by PTF. A load factor of unity is applied to permanent loads for the fatigue limit state only when specified.

²Rating applicable for box culverts only

³Minimum ES of 0.50 applies for top slabs of box culverts

⁴See A3.4.2, Load Factors for Construction Loads

	Box Culvert Earth Load Factors	
	Maximum	Minimum
γ_{EV}	1.30	0.90
γ_{EH}	1.35	0.90*
*Use 0.50 minimum for culvert top slab		

Table 3.4.1.1P-5 - Load Factors and Live Load Vehicles for Steel Floorbeams

	Load Combination												
	STR I	STR IP ⁷	STR IA ⁵	STR II	STR III	STR V	SERV II	SERV IIA ⁵	SERV IIB	Fatigue I ¹ (infinite)	Fatigue II ¹ (finite)	DEFL ¹	CONST/ UNCURED SLAB ⁸
γ_{DC} ²	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.00	1.00	1.00	---	---	---	1.25
γ_{DW} ³	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.00	1.00	1.00	---	---	---	1.50, 0.65
γ_{LL}	1.75	1.35	1.35	1.35	---	1.35	1.30	1.00	1.00	1.50	0.75	1.00	1.50
γ_{PL}	---	1.75	---	---	---	---	---	---	---	---	---	---	---
γ_{WS}	---	---	---	---	1.40	0.40	---	---	---	---	---	---	1.25
Design LL Veh. ⁶	PHL-93	PHL-93	PHL-93	Permit (P-82)	---	PHL-93	PHL-93	PHL-93	Permit (P-82)	HS20- 30	HS20- 30	PennDOT Defl. Trk.	User Def.
Rating Veh.	Rating Applicability: I = Inventory, O = Operating												
PHL-93	I	I	O	---	---	---	I	O	---	---	---	---	---
P-82	---	---	---	O	---	---	---	---	O	---	---	---	---
ML-80	I	I	---	O	---	---	I	O	---	---	---	---	---
TK527	I	I	---	O	---	---	I	O	---	---	---	---	---
HS20	I	I	---	O	---	---	I	O	---	---	---	---	---
H20	I	I	---	O	---	---	I	O	---	---	---	---	---
Spec. Veh.	I	I	---	O	---	---	I	O	---	---	---	---	---

Table 3.4.1.1P-5 - Load Factors and Live Load Vehicles for Steel Floorbeams (Continued)

Notes:

¹A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified²DC load factor also used for barrier loads, sidewalk, median barrier, railings, etc.³DW load factor also used for utility loads⁴All loads applied to non-composite section for non-composite floorbeams (Live loads are applied to the n section for steel)⁵Load combination for rating only⁶This row lists the typical design vehicle to be used for each load combination⁷The reduced load factor for LL with PL (see D3.4.1)⁸Live load N/A for uncured slab check

Permanent Loads for Floorbeam Programs			Section Properties ⁴
Load	Steel		Steel
DC1	γ_{FLBM}	Floorbeam	nc
	γ_{SLAB}	Slab	nc
	γ_{SLAB}	Haunch	nc
DC2	γ_{DC2}	Barrier, Sidewalk, Median Barrier, Railings, etc.	3n
DW	γ_{FWS}	FWS	3n

Table 3.4.1.1P-6 - Load Factors and Live Load Vehicles for Steel Trusses

	Load Combination															
	STR I	STR IP ⁸	STR IA ⁵	STR II	STR III	STR IV ¹	STR V	EXT. EVENT III	EXT. EVENT IV	SERV II	SERV IIA ⁵	SERV IIB	Fatigue I ² (infinite)	Fatigue II ² (finite)	DEFL ²	CONST
γ_{DC} ³	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.25, 0.90	1.5	1.25, 0.90	1.25, 0.90	1.05, 0.95	1.00	1.00	1.00	---	---	---	1.25
γ_{DW} ⁴	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.50, 0.65	1.05, 0.90	1.00	1.00	1.00	---	---	---	1.50, 0.65
γ_{LL}	1.75	1.35	1.35	1.35	---	---	1.35	1.30	1.15	1.30	1.00	1.00	1.50	0.75	1.00	1.50
γ_{PL}	---	1.75	---	---	---	---	---	1.10	1.05	---	---	---	---	---	---	---
γ_{WS}	---	---	---	---	1.40	---	0.40	---	---	---	---	---	---	---	---	1.25
Design LL Veh. ⁶	PHL-93	PHL-93	PHL-93	Permit (P-82)	---	---	PHL-93	PHL-93 P-82	PHL-93 P-82	PHL-93	PHL-93	Permit (P-82)	HS20-30	HS20-30	PennDOT Defl. Trk.	User Def.
Rating Veh.	Rating Applicability: I = Inventory, O = Operating															
PHL-93	I	I	O	---	---	---	---	---	---	I	O	---	---	---	---	---
P-82	---	---	---	O	---	---	---	---	---	---	---	O	---	---	---	---
ML-80	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---	---
TK527	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---	---
HS20	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---	---
H20	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---	---
Spec. Veh.	I	I	---	O	---	---	---	---	---	I	O	---	---	---	---	---

Table 3.4.1.1P-6 - Load Factors and Live Load Vehicles for Steel Trusses (Continued)

Notes:

¹Applicable when DL/LL ratio exceeds 7.0

²A load factor of unity is applied to permanent loads for the fatigue and deflection limit state only when specified

³DC load factor also used for barrier loads, sidewalk, median barrier, railings, deck, stringers, truss floorbeams, wind and lateral bracing, etc.

⁴DW load factor also used for utility loads

⁵Load combination for rating only

⁶This row lists the typical design vehicle to be used for each load combination

⁷The reduced load factor for LL with PL (see D3.4.1)

SPECIFICATION

COMMENTARY

3.5 PERMANENT LOADS**3.5.1 Dead Loads: DC, DW and EV**

The following shall supplement A3.5.1.

In addition to the weight of the deck slab, the design dead load shall include provisions for a future wearing surface dead load of 0.030 ksf on the deck slab between the curbs. This load shall be considered for all deck slabs, including decks with a bituminous wearing surface, but not for structures under fill. For decks formed using permanent metal deck forms, an additional dead load shall be included based on 0.015 ksf which takes into account the weight of the form, plus the weight of the concrete in the valleys of the forms.

In Table A3.5.1-1, replace the lightweight concrete value of 0.110 kcf with 0.115 kcf. Also in Table A3.5.1-1, delete the "sand-lightweight" concrete value. For use of lightweight concrete with unit weights different than 0.115 kcf, see D5.4.2.1 and DC5.4.2.1.

3.5.1.1P Application of Dead Load on Girder and Box Beam Structures

The provisions in this article apply to superstructure types, a, b, c, f, g, h, k and l given in Table A4.6.2.2.1-1.

For composite adjacent and spread beams, the barrier (single barrier) load shall be equally distributed to the nearest three (adjacent) and two (spread) beams, respectively, when the barriers are placed after slab has hardened.

The dead load of items, such as fencing and sound barriers, if placed after the slab has hardened, shall be distributed to girder as described above.

Sidewalk dead load shall be distributed to a girder using the lever rule.

For noncomposite girders, the barrier load shall be distributed solely to the fascia girder and the sidewalk dead load shall be distributed as indicated above.

The future wearing surface shall be distributed equally among all girders.

3.6 LIVE LOADS**3.6.1 Gravity Loads: LL, PL****C3.5.1**

The following shall supplement AC3.5.1.

The normal weight concrete and lightweight concrete with unit weights of 0.150 kcf and 0.115 kcf respectively include an allowance for reinforcement in the calculation of the weight.

For concrete deck slabs, provisions must be made in the design for the addition of a bituminous wearing surface at some future time. Even in cases where the initial design includes a bituminous surface, provision must be made for an additional future wearing surface since the original bituminous material is not always stripped off before the new surface is added.

For structures under fill, the additional dead load associated with a future wearing surface is insignificant when compared with other contributions to the dead load. Therefore, in this case, no allowance for future wearing surface is necessary.

It is recognized that permanent metal deck forms are available for which the dead load is less than 0.015 ksf; however, the minimum design load should be retained at this level. Lightweight forms may be advantageous in certain situations, such as rehabilitation, and should be evaluated on a case-by-case basis.

The use of preformed cellular polystyrene ("Styrofoam") in the valleys of the metal deck forms to reduce the dead load is permitted on rehabilitation projects.

SPECIFICATION

COMMENTARY

3.6.1.1 Vehicular Live Load*3.6.1.1.2 Multiple Presence of Live Load*

Delete the last sentence of the second paragraph of A3.6.1.1.2.

C3.6.1.1.2

Delete the third through seventh paragraphs of AC3.6.1.1.2.

3.6.1.2 Design Vehicular Live Load*3.6.1.2.1 General*

The following shall replace the first paragraph of A3.6.1.2.1.

The vehicular live loading on the roadways of bridges or incidental structures, designated PHL-93, shall consist of a combination of the:

- design truck or design tandem, and
- design lane load,

as given in A3.6.1.2 and D3.6.1.2.

C3.6.1.2.1

The following shall supplement AC3.6.1.2.1.

At this time, the Department makes no exceptions to the requirements for application of PHL-93 vehicular live load for bridges on low volume roads.

3.6.1.2.3 Design Tandem

Modify A3.6.1.2.3 so that the weight of each axle is increased from 25 kips to 31.25 kips.

*3.6.1.2.5 Tire Contact Area**C3.6.1.2.5*

The following shall replace AC3.6.1.2.5.

The area load applies only to the design truck and tandem. For other design vehicles, the tire contact area should be determined by the engineer.

As a guideline for other truck loads, the tire area in in.² may be calculated from the following dimensions:

Tire width = $P/0.8$

Tire length = $6.4\gamma(1+IM/100)$

where:

γ = load factor, as given in A3.4.1 and D3.4.1, except for buried structures where the load factor shall be 1.35

IM = dynamic load allowance percent

P = wheel load
= 16 kips for the design truck, 15.625 kips for the design tandem and 13.5 kips for the P-82

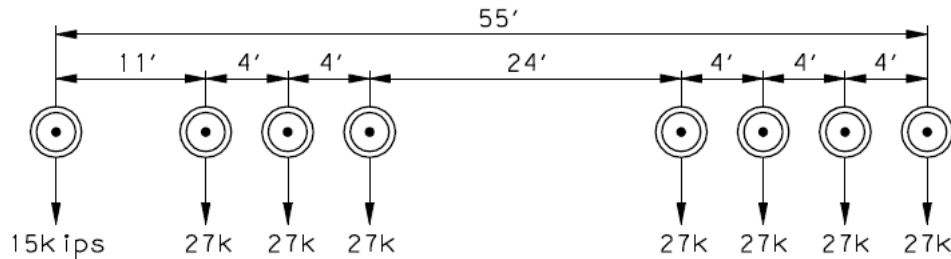
A constant value of γ was chosen for buried structures to simplify the analysis at strength and service limit states. PennDOT has conducted a study to ensure that the use of a constant load factor has a negligible effect.

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COMMENTARY

3.6.1.2.7P *Design Permit Load (P-82)*

The weights and spacings of axles and wheels for the Permit Load (P-82) shall be as specified in Figure 3.6.1.2.7P-1. A dynamic load allowance shall be considered as specified in D3.6.2 and A3.6.2.



NOTE: P-82 width is the same as the Design Truck.
Transverse wheel location is the same as Design Truck.

Figure 3.6.1.2.7P-1 - Pennsylvania Permit Load (P-82) 102 tons, 8 Axle

Axles which do not contribute to the extreme force effect under consideration shall be neglected.

For multi-girder superstructures design, the permit load shall be in one lane or in multiple lanes whichever is the controlling case.

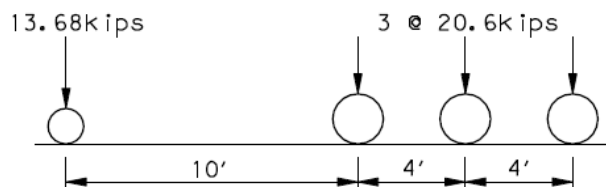
For superstructure with girder-floorbeam-stringer systems and substructure components designs, the permit load shall be in one lane or in one lane with PHL-93 loading in adjacent lanes, whichever is the controlling case.

3.6.1.2.8P *Maximum Legal Load (ML-80)*

C3.6.1.2.8P

The weights and spacings of axles and wheels for the Maximum Legal Load (ML-80) shall be as specified in Figure 3.6.1.2.8P-1. The ML-80 truck is used for rating; however, the inventory rating shall be 1.0 or greater for new designs.

The ML-80 is not considered a notional load. Therefore, all of the axles shall be considered when determining force effects.



NOTE: ML-80 width is the same as the design truck.
Transverse wheel location is the same as design truck.

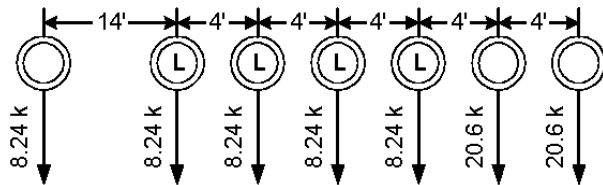
Figure 3.6.1.2.8P-1 - Pennsylvania Maximum Legal Load (ML-80) 37.74 tons, 4 Axle

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COMMENTARY

3.6.1.2.9P PA Legal Load Configuration (TK527)

The axle weights and spacings to be used to analyze and rate bridges for the PA Legal Load Configuration (TK527) shall be as specified in Figure 3.6.1.2.9P-1. The TK527 truck is used for rating; however, the inventory rating shall be 1.0 or greater for new designs.



NOTE: GVW = 80,000 lbs + 3% scale tolerance
Gage Distance = 6', L indicates Lift Axle

Figure 3.6.1.2.9P-1 - TK527 Vehicle for Rating

3.6.1.3 Application of Design Vehicular Live Loads

3.6.1.3.1 General

The following shall replace A3.6.1.3.1.

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load, or
- The effect of one design truck with the variable axle spacing specified in A3.6.1.2.2, combined with the effect of the design lane load, and
- For the negative moment between points of dead load contraflexure, the effect of two design trucks spaced a minimum of 50 ft. between the lead axle of one truck and the rear axle of the other truck, combined with the effect of the design lane load; the distance between the 32 kips axles of each truck shall be taken as 14 ft.. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

For the reaction at interior piers only, 90% of the effect of two design trucks spaced a minimum of 50 ft. between the lead axle of one truck and the rear axle of the other truck, combined with 90% of the effect of the design lane load. The distance between the 32 kips axles of each truck shall be taken as 14 ft.. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

- For the negative moment between points of dead load contraflexure, the effect of two tandems with axle

C3.6.1.3.1

Delete the second and fifth sentences of the third paragraph of AC3.6.1.3.1.

The following shall supplement AC3.6.1.3.1.

The BXLRFD program does not consider the effect of two design trucks, since the minimum distance between the two design trucks is 50 ft. which is at the upper limit of a twin cell culvert. The effects of two tandems are considered for a twin cell box culvert in the BXLRFD program.

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weights of 25 kips spaced from 26 ft. to 40 ft. apart, combined with the effect of the design lane load. The two tandems should be placed in adjacent spans to produce maximum force effects.

For the reaction at interior piers only, 100% of the effect of two tandems with axle weights of 25 kips spaced from 26 ft. to 40 ft. apart combined with the effect of the design lane load. The two tandems should be placed in adjacent spans to produce maximum force effects.

Axles which do not contribute to the extreme force effect under consideration shall be neglected.

Both the design lanes and the position of the 10 ft. loaded width in each lane shall be positioned to produce extreme force effects. The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than:

- For the design of the deck overhang - 1 ft. from the face of the curb or railing, and
- For the design of all other components - 2 ft. from the edge of the design lane.

Unless otherwise specified, the lengths of design lanes, or parts thereof, which contribute to the extreme force effect under consideration, shall be loaded with the design lane load.

3.6.1.3.2 Loading for Live Load Deflection Evaluation

C3.6.1.3.2

The following shall replace A3.6.1.3.2.

The deflection should be taken as 125% of the larger of:

- that resulting from the effect of one design truck with the variable axle spacing specified in A3.6.1.2.2,
- that resulting from the effect of 25% of one design truck with the variable axle spacing specified in A3.6.1.2.2, combined with the effect of the design lane.

The following shall replace AC3.6.1.3.2.

The LRFD live load deflection criteria was developed such that deflections would be roughly equivalent to those produced by a HS20 vehicle. A 25% increase is specified to be consistent with the Department's past use of the HS25 vehicle for computing deflections.

3.6.1.3.3 Design Loads for Decks, Deck Systems, and the Top Slab of Box Culverts

Replace the two bullets of the second paragraph of A3.6.1.3.3 with the following.

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of A3.6.1.2.2 or design tandem of D3.6.1.2.3 shall be applied to the deck slab or top slab of box culverts.
- Where the slab spans primarily in the longitudinal

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direction:

- For top slabs of box culverts of all spans and for all other cases, including slab-type bridges where the span does not exceed 15.0 ft., only the axle loads of the design truck or design tandem of A3.6.1.2.2 and D3.6.1.2.3, respectively, shall be applied.
- For all other cases, including slab-type bridges (excluding top slabs of box culverts) where the span exceeds 15.0 ft., all of the loads specified in D3.6.1.2 shall be applied.

Replace the third paragraph of A3.6.1.3.3 with the following:

Where the refined methods are used to analyze decks, force effects shall be determined on the following basis:

- Where the slab spans primarily in the transverse direction, only the axles of the design truck of A3.6.1.2.2 or design tandem of D3.6.1.2.3 shall be applied to the deck slab.
- Where the slab spans primarily in the longitudinal direction (including slab-type bridges), all of the loads specified in D3.6.1.2 shall be applied.

3.6.1.3.4 Deck Overhang Load

The following shall replace A3.6.1.3.4.

The deck overhang load shall be as given in D3.6.1.3.1.

Also, the ultimate strength of the deck section shall be greater than the ultimate strength of the barrier, see Section A13 and its Appendix. Horizontal loads on the overhang resulting from vehicle collision with barriers shall be in accordance with the provisions of Section A13 and its Appendix.

3.6.1.4 Fatigue Load

3.6.1.4.2 Frequency

The following shall replace Table A3.6.1.4.2-1.

Table 3.6.1.4.2-1 – Fraction of Truck Traffic in a Single Lane, p

Number of Lanes Available to Trucks	p
1	1.00
2 or more	0.85

C3.6.1.3.4

The following shall replace AC3.6.1.3.4.

The deck overhang slab provided in BD-601M has been designed for the vertical design loads (D3.6.1.3.1) or a strength greater than the applied forces transmitted to the overhang when the barrier is subjected to the maximum collision force it can resist (Section A13) whichever is greater. The ultimate strength of the barrier used in the design of the overhang was based on the Department's Typical Concrete Barrier (see Standard Drawing BD-601M) which placed greater demand on the deck overhang than the other Department barriers.

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3.6.1.5 Rail Transit Load*3.6.1.5.1P General*

Live loads for rail traffic shall use a combination of axle loads and axle spacings represented by the Cooper E80 loading, as shown in Figure D3.6.1.5.1P-1.

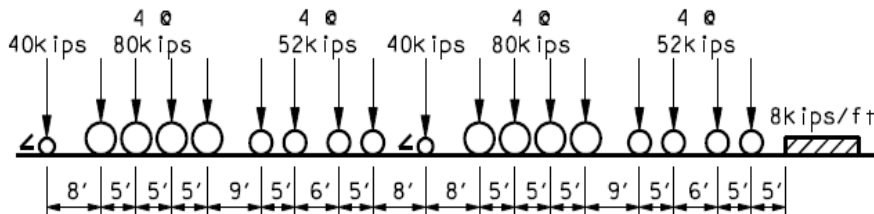


Figure 3.6.1.5.1P-1 - Wheel Spacing for Cooper E80 Design Loading (Load/Axle)

3.6.1.5.2P Distribution of Rail Transit Loads Through Earth Fill

The load intensity, W_l , on a buried structure due to rail transit loading shall be determined using the following relationship:

$$W_l = C P_o B_c (1 + I_i) \quad (3.6.1.5.2P-1)$$

Refer to Table 3.6.1.5.2P-1 for values of C . The series of axle loads and spacing shall be converted into a uniform load at the bottom of the railroad ties. The loading, P_o , at the base of the ties shall be represented by a ground pressure of 2025 ksf, which represents the locomotive drive-wheel (four at 80 kips) loading distributed over an area 8 ft. by 20 ft. and a track structure loading of 0.2 kip/ft. The impact factor, I_i , shall range from 40% at zero cover to 0% at 10 ft. of cover.

The live load and the dead load, including the impact factor, for a Cooper E80 loading can be determined from Figure 3.6.1.5.2P-1. To obtain the live load per linear foot, multiply the unit load from Figure 3.6.1.5.2P-1 by the outside horizontal span of the pipe, B_c .

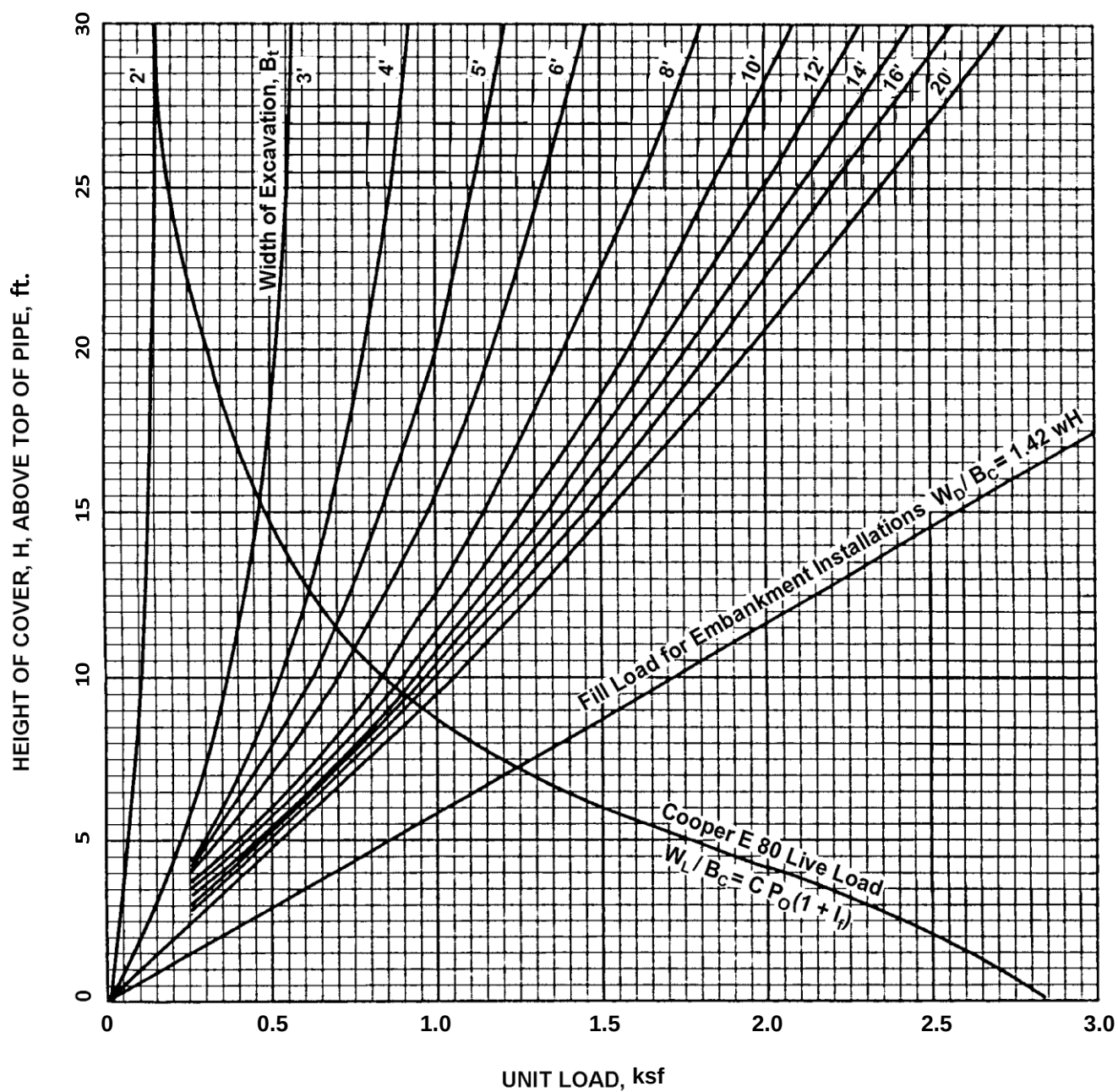
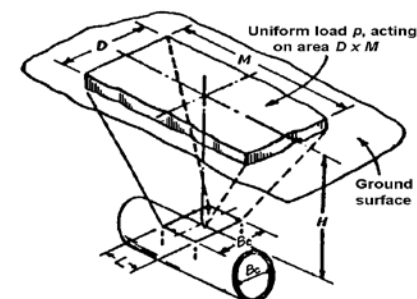


Figure 3.6.1.5.2P-1 - Live and Dead Loads on Pipe Installed Under Railroads (ACPA, 1981)

Table 3.6.1.5.2P-1 - Values of Load Coefficient (C) for Concentrated and Distributed Superimposed Loads Vertically Centered Over Culvert (ASCE, 1969)

$\frac{D}{2H}$ or $\frac{B_c}{2H}$	$\frac{M}{2H}$ or $\frac{L}{2H}$													
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	5.0
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.121	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.211	0.218
0.3	0.053	0.103	0.149	0.190	0.221	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.281	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.484	0.505	0.525	0.540	0.548
0.6	0.089	0.171	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.516	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.581	0.615	0.639	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.766	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.819	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.891	0.916
2.0	0.121	0.211	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.819	0.894	0.930	0.956

*Influence coefficients for solution of Holl's and Newmark's integration of the Boussinesq equation for vertical stress



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3.6.1.6 Pedestrian Loads

The following shall replace A3.6.1.6.

A pedestrian load of 0.075 ksf shall be applied to all sidewalks wider than 2.0 ft. and considered simultaneously with the vehicle design live load.

The pedestrian load is distributed using the lever rule.

When the pedestrian load is required, two loading conditions shall be considered. The first loading condition assumes that the sidewalk is not present (i.e., an extended roadway surface and barrier would replace the sidewalk area) and the bridge is used for vehicular live load only. Under the second loading condition, the pedestrian load is present and the vehicular live load is factored at a reduced level. The Strength IP load combination was developed for the second loading condition.

Bridges intended for only pedestrian, equestrian, light maintenance vehicles, and/or bicycle traffic should be designed in accordance with AASHTO's LRFD *Guide Specifications for the Design of Pedestrian Bridges*.

3.6.2 Dynamic Load Allowance: IM**3.6.2.1 General**

The following shall supplement A3.6.2.1.

For permit loads, the static effect of the P-82 shall be increased by a percentage not to exceed $IM = 20\%$.

IM for deck design = 50%

The second to last paragraph in A3.6.2.1 which begins "Dynamic load allowance need not..." shall be deleted.

3.6.2.1.1P Components for which IM is Applicable

The following components shall have the IM factor included in the design:

- all superstructure components including deck and deck joints
- pier caps and shafts
- backwalls and pedestals of abutments
- bearings, except for plain and reinforced elastomeric bearings

For buried components covered in D12 and A12, see D3.6.2.2.

C3.6.2.1

The following shall supplement AC3.6.2.1.

Irregularities in decks such as potholes can result in large localized impact effects. As a result, PennDOT requires that the impact for decks be increased from 33% to 50%. Other elements of the bridge structure should not be greatly affected by high localized impact due to dampening. The combination of 50% impact, the design truck (former HS20 truck) and LRFD deck design criteria will produce deck designs comparable to 30% impact, HS25 and past AASHTO deck design criteria.

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3.6.2.1.2P Components for which IM is Not Applicable

The following components shall not have the IM factor included in the design:

- retaining walls not subject to vertical reactions from the superstructure, including MSE walls
- foundation components which are entirely below ground level, including footings (except for frame and box culverts where IM is applicable as per D3.6.2.2), piles, caissons and pedestals
- abutment stems
- plain and reinforced elastomeric bearings
- buried components with 8 ft. or greater fill above them (see D3.6.2.2)

The pedestrian load shall not have the IM factor applied.

3.6.2.2 Buried Components

The following shall replace A3.6.2.2.

The dynamic load allowance for culverts and other buried structures covered by Section 12, in percent, shall be taken as:

$$IM = 40 (1.0 - 0.125 D_E) \geq 0\% \quad (3.6.2.2-1)$$

where:

D_E = the minimum depth of earth cover above the structure (ft.)

Dynamic load allowance shall not be applied to foundation pressures.

3.6.4 Braking Force: BR

The following shall supplement A3.6.4.

Dynamic load allowance is not applied to the braking force.

3.6.5 Vehicular Collision Force: CT**3.6.5.2 Vehicular Collision with Barriers**

The following shall supplement A3.6.5.3.

For transverse vehicular collision loading transferred to the substructure for u-wings and retaining walls, use a load

COMMENTARY

C3.6.2.1.2P

The PAPIER and VBent programs carries the live loads from the pier cap through to the footing without the removal of the effect of the dynamic load allowance (IM) input by the user. This provides a consistent mathematical model throughout the structure, where the moments, shears, and axial forces at the bottom of the column are equal to those at the top of the footing of the pier.

C3.6.4

The following shall supplement CA3.6.4.

LRFD analysis of the capacity of existing substructure units on shorter span bridges may become problematic. Use of the original design braking force, requiring approval of the Chief Bridge Engineer, may be warranted for analysis of these older structures.

C3.6.5.2P

The transverse vehicular collision loading of 10 kips acting over 5 ft may be distributed down to the footing at a 1:1 slope. Adjacent to open joints, this load may only be

SPECIFICATION

of 10 kips acting over 5 ft length applied at a distance equal to the height of the concrete barrier above the top of the wall.

3.7 WATER LOADS: WA**3.7.1 Static Pressure**

The following shall replace the second paragraph of A3.7.1.

The design high water level shall be based on the scour design flood as defined in PP7.2.2. The water loads utilized in the design of piers, abutments and other bridge components shall be based on the appropriate water levels (high, normal {mean} and low) and considered at the strength and service limit states. The design of piers shall consider a low water level condition where a large portion of the pier is below the normal water level. The maximum depth of scour shall be based on either the scour design flood or the superflood as defined in PP7.2.2 or PP7.2.3 respectively. Analysis of the structure using the maximum scour depth, shall utilize the scour design flood water level and water loads and shall be considered at the Service I limit state.

3.7.5 Change in Foundations Due to Limit State for Scour

The following shall replace the second paragraph of A3.7.5.

The consequences of changes in the foundation conditions from the scour design flood (100-year, record or lesser overtopping flood) which results in the worst-case scour condition shall be considered at the strength and service limit states. The consequences of changes in the foundation conditions due to scour resulting from a superflood, as defined in PP 7.2.3, shall be considered at the Service I limit state based on the Department's preference for the bridge to remain operational following a severe flood event.

3.8 WIND LOAD: WL AND WS**3.8.1 Horizontal Wind Pressure****3.8.1.2 Wind Pressure on Structures: WS***3.8.1.2.1 General*

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distributed in one direction which will usually be the controlling condition. Distributing the load in one direction is conservative for footing designs, since the footings are continuous at open joints.

C3.7.1P

The low water level condition is used to check the pier with an increased exposure to wind loads combined with a reduced water level and water loads.

Water levels associated with floods exceeding the scour design flood, may be evaluated at the extreme event II limit state using the corresponding water level, water loads and scour depth.

C3.7.5

The following shall supplement AC3.7.5.

The stability of pile foundations exposed during a severe flood event (superflood) shall be investigated to verify the unsupported length of the axially loaded piles will not fail under design loads using water elevation and pressure resulting from the scour design flood while the extent of the damage is unknown.

C3.8.1.2.1

The following shall replace the third paragraph of AC3.8.1.2.1.

If approved by the Chief Bridge Engineer, wind tunnel tests may be used to provide more precise estimates of wind pressures. Such testing should be considered where wind is a major design load.

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3.8.1.2.2 *Loads from Superstructures*

C3.8.1.2.2

The following shall supplement AC3.8.1.2.2.

The columns referenced in A3.8.1.2.2 are columns in arch bridges not columns in substructure units.

3.8.3 Aeroelastic Instability

3.8.3.4 Wind Tunnel Tests

The following shall replace A3.8.3.4.

If approved by the Chief Bridge Engineer, representative wind tunnel tests may be used to satisfy the requirements of A3.8.3.2 and A3.8.3.3.

3.9 ICE LOADS: IC

3.9.1 General

The following shall supplement A3.9.1.

The forces due to ice shall be applied at the average elevation of the highest expected water elevation and the normal water elevation.

C3.9.1

The following shall supplement AC3.9.1.

The PAPIER and VBent programs use a default ice thickness of 6 in. and a default ice crushing strength of 58 ksf.

A study prepared for the Department, as part of the Foxburg Bridge Replacement project, recommended an ice thickness of 1.9 feet (22.8 in.) and an ice crushing strength of 170 psi (24.48 ksf) be used at the project site. The lower ice crushing strength combined with the increased ice thickness resulted in a horizontal ice force that was approximately two times larger than the calculated ice force in PAPIER, using the default values for ice crushing strength and ice thickness. The 6 in. ice thickness and 58 ksf ice crushing strength in PAPIER and VBent is approximately equivalent to the AASHTO ice crushing strengths with the following ice thicknesses:

- 8 ksf with an ice thickness equal to 29.61"
- 16 ksf with an ice thickness equal to 17.42"
- 24 ksf with an ice thickness equal to 12.6"
- 32 ksf with an ice thickness equal to 9.95"

3.9.5 Vertical Forces due to Ice Adhesion

The following shall replace A3.9.5.

The vertical force on a bridge pier due to rapid water level fluctuation shall be taken as:

- for a circular pier, in kips:
 $6.27 t^2 + 1.48 R t^{1.25}$ (3.9.5-1)
- for an oblong pier, in kips:
 $6.27 t^2 + 1.48 R t^{1.25} + 0.2 L t^{1.25}$ (3.9.5-2)

where:

t = ice thickness (ft.)

C3.9.5

Delete AC3.9.5.

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R = radius of circular pier (ft.) or approximated end of oblong pier

L = perimeter of pier excluding half circles at ends of oblong pier (ft.)

3.10 EARTHQUAKE EFFECTS: EQ**3.10.1 General****C3.10.1**

The following shall supplement AC3.10.1.

Minimize bridge skew as much as and whenever possible. It is well known that skewed structures perform poorly in seismic events when compared to the performance of normal or non-skewed structures.

These specifications present seismic design and construction requirements applicable to the majority of highway bridges to be constructed in the United States. Bridges not covered by these provisions probably constitute 5 to 15 percent of the total number of bridges designed.

The Project Engineering Panel (PEP) of the Applied Technology Council (ATC) has decided that special seismic design provisions are not required for buried structures. It was recognized by the PEP, however, that this decision may need to be reconsidered as more research data on the seismic performance of this type of structure become available.

These specifications specify minimum requirements. More sophisticated design or analysis techniques may be utilized if deemed appropriate by the Design Engineer.

For bridge types not covered by these specifications, the following factors should be considered.

- (a) The recommended elastic design force levels of the specifications should be applicable because force levels are largely independent of the type of bridge structure. It should be noted that the elastic design force levels of the specifications are part of a design philosophy described in Chapter 1 of FHWA Research Report FHWA-IP-87-6. The appropriateness of both the design force levels and the design philosophy must be assessed before they are used for bridges that are not covered by these specifications.
- (b) A multi-mode dynamic analysis should be considered, as specified in A4.7.4.3.3.
- (c) Design displacements are as important as design forces; when possible, the design methodology should consider displacements arising from the effects discussed in AC4.7.4.4.
- (d) If a design methodology similar to that used in these specifications is deemed desirable, the design requirements contained herein should be used to

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COMMENTARY

ensure compliance with the design philosophy.

Use caution when referencing the flowcharts contained in the Appendix to A3. They reference AASHTO LRFD Specifications which may be modified by the Design Manual, Part 4.

3.10.3 Site Effects

The following shall supplement A3.10.3.

As part of the Foundation submission, the designer shall indicate if the site soils are classified as either Site Class E or F.

The primary method to determine Site Class E soil is based on the average Standard Penetration Test blow count, N value, for the upper 100 feet of the soil profile.

For structures within Site Class E or F soils, the transverse reinforcement requirements at the top and bottom of a column shall be as specified in A5.10.11.4.1d and A5.10.11.4.1e.

3.10.6 Seismic Performance Zones

The following shall supplement A3.10.6.

Based on the General Procedure, Pennsylvania is classified as Seismic Zone 1.

3.10.9 Calculation of Design Forces**3.10.9.2 Seismic Zone 1**

The following shall replace the first and second paragraph of A3.10.9.2.

For all structures in Pennsylvania, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load.

3.10.9.5 Longitudinal Restrainers

The following shall supplement A3.10.9.5.

Restrainers may only be used with the prior approval of the Chief Bridge Engineer.

3.11 EARTH PRESSURE: EH, ES, LS AND DD**3.11.1 General**

The following shall supplement A3.11.1.

Both the vertical and horizontal components of an inclined lateral earth pressure shall be considered for application of load and load factors.

C3.10.3

The following shall supplement AC3.10.3.

For Site Class E soils, a Standard Penetration Test blow count of greater than 10 should result in undrained shear strength greater than 1.0 ksf.

For site conditions that meet the following, the designer shall obtain appropriate soil samples to test the undrained shear strength:

- Boring located at a pier
- Standard Penetration Test blow count is less than 10
- Clay thickness is greater than 10 feet

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3.11.3 Presence of Water

The following shall supplement A3.11.3.

Walls along a stream or river shall be designed for a minimum differential water pressure due to a 3'-0" head of water in the backfill soil above the weephole inverts.

3.11.5 Earth Pressure: EH**3.11.5.2 At-Rest Lateral Earth Pressure Coefficient, k_0** **3.11.5.3 Active Lateral Earth Pressure Coefficient, k_a**

COMMENTARY

C3.11.3

The following shall supplement AC3.11.3.

Evaluation of water pressures and seepage forces is critical in the design of retaining walls because water pressures and seepage forces are the most common causes of retaining wall failure. Seepage forces and water pressures affect the stability of retaining walls by:

- Increasing the weight of soil behind the wall through saturation, thereby increasing the driving soil pressure
- Decreasing the effective weight of soil in front of the wall through upward seepage forces, thereby reducing the resisting soil pressure
- Decreasing the effective stress (normal force) on the wall foundation due to wall weight through uplift, thereby reducing sliding resistance and resistance to overturning.

C3.11.5.2

The following shall supplement AC3.11.5.2.

At-rest earth pressures are usually limited to bridge abutments to which superstructures are fixed prior to backfilling (e.g., framed bridges) or to cantilevered walls where the heel is restrained and the base/stem connection prevents rotation of the stem.

C3.11.5.3

The following shall supplement AC3.11.5.3.

The differences between the Coulomb Theory currently specified, and the Rankine Theory specified in the past is illustrated in Figure AC3.11.5.3-1. The Rankine theory is the basis of the equivalent fluid method of A3.11.5.5 and the design procedures for mechanically stabilized earth walls.

Gravity and semi-gravity walls usually deflect a sufficient amount during backfilling to develop an active state of stress in the retained soil. This also is true of cantilevered and counterfort walls unless the heel is tied down or otherwise restrained and the base/stem connection prevents sufficient rotation of the stem to develop an active state of stress in the soil.

Wall movements cause the development of friction between the wall and the soil in contact with the wall. This resulting frictional force has the effect of inclining the earth pressure resultant on the wall, whereas the resultant would be normal to the wall in the case of no friction. The angle of inclination of the earth pressure resultant with respect to a line normal to the wall is called the angle of wall friction (δ).

SPECIFICATION

3.11.5.5 Equivalent-Fluid Method of Estimating Rankine Lateral Earth Pressures

The following shall supplement A3.11.5.5.

Cohesionless soils with a maximum fines content of 5% by weight shall be used for backfill. This criteria can be met by backfilling with AASHTO No. 57 or the Department's open graded subbase (OGS) in conformance with Publication 408, Section 703.

For yielding walls backfilled with these materials, the design earth pressure at any depth shall be defined as increasing at a rate of 0.035 ksf/ft, plus the live load surcharge from A3.11.6.4 and D3.11.6.4.

For unyielding walls, restrained abutments (e.g., backfilled after superstructure erection), at-rest earth pressure, increasing at 0.050 ksf/ft, plus the live load surcharge from A3.11.6.4 and D3.11.6.4, shall be used.

The following shall supplement A3.11.5.5 for the design of box culverts.

For box culverts, equivalent fluid density shall be taken as specified in Table 3.11.5.5P-2.

Table 3.11.5.5-2P - Equivalent Fluid Densities for Box Culverts

	Level Backfill (kef)	Backfill with $\beta=25^\circ$ (kef)
Minimum	0.045	0.055
Maximum	0.070	0.080

These equivalent fluid densities along with the appropriate maximum and minimum load factors shall be selected to produce the extreme force effects.

3.11.5.6 Lateral Earth Pressures for Nongravity Cantilevered Walls

The following shall supplement A3.11.5.6.

Some portion of the embedded depth below finished grade, noted as the difference between the finished grade and the design grade in Figures A3.11.5.6-1 through A3.11.5.6-5, (usually 3 ft. for an element in soil, and 1 ft. for an element in rock) is ineffective in providing passive lateral support.

In developing the design lateral pressure, the lateral pressure due to water, live load surcharge, permanent point and line surcharge loads, backfill compaction, or other types

COMMENTARY

C3.11.5.5

In the fifth paragraph of AC3.11.5.5, remove the reference to Figure AC3.11.5.3-1.

The following shall supplement AC3.11.5.5.

Soils with more than 5% fines shall be avoided as backfill because of their low permeability and potential frost susceptibility.

For design, the Department's open graded subbase (OGS) shall have the following assumed properties:

- moist density = 0.120 kef
- saturated density = 0.135 kef
- angle of internal friction = 30°

The following shall supplement AC3.11.5.5 for the design of box culverts.

Two soil types were selected for design to reflect potential lateral at-rest earth pressures for box culverts, considering construction practice and soil variability in Pennsylvania. The engineered backfill required for a distance of only 1 ft. from the face of the culvert wall is not sufficient to reduce lateral earth pressures to levels that would be expected for abutments and retaining walls for which more detailed backfill requirements are specified. Lateral earth pressures resulting from the factored load combinations, specified in this article, A3.4.1 and D3.4.1 compare closely with past DM-4 practice.

Although the equivalent fluid weights given in Table A3.11.5.5-1 correspond to those for "Dense Sand or Gravel" and "Compacted Lean Clay", backfill material shall be in conformance with the requirements given in Publication 408 and the contract documents. Equivalent fluid weights specified herein are for design only. Values of the equivalent fluid pressure for a sloping backfill are provided for the rare case in which the culvert is parallel to the roadway. In such a case, consideration should be given to sliding as a result of the imbalance of lateral loads.

C3.11.5.6

The following shall supplement AC3.11.5.6.

The upper 2 to 3 ft. of the discrete embedded vertical element in soil, or 1 ft. in rock, is typically assumed ineffective in mobilizing passive resistance to account for the effects of freezing and thawing, weathering or other shallow ground disturbance (e.g., utility excavations or pavement replacement in front of the wall).

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of surcharge loads shall be added to the lateral earth pressure.

3.11.5.7 Apparent Earth Pressure (AEP) for Anchored Walls

3.11.5.7.1 Cohesionless Soils

The following shall supplement A3.11.5.7.1:

The apparent earth pressure distribution for temporary and permanent anchored walls constructed from the top down and supporting cohesionless soil may be determined using Figure A3.11.5.7.1-1. Water pressures and surcharge pressures, if applicable, should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall. Determine geostatic water pressure on the wall using the maximum expected water table differential between excavation interior and exterior, based on borings or other information.

3.11.5.7.2 Cohesive Soils

The following shall supplement A3.11.5.7.2:

Use the undrained shear strength of the soil through which the excavation extends.

3.11.5.7.2a Stiff to Hard

The following shall supplement A3.11.5.7.2a:

The apparent earth pressure distribution for temporary anchored walls constructed from the top down and supporting stiff to hard cohesive soils ($N_S \leq 4$) including fissured clays, where temporary conditions are of a controlled short duration and for which there is no available free water, may be determined using Figure A3.11.5.7.1-1.

Surcharge pressures, if applicable, should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall.

For any case, surcharge pressures, if applicable,

COMMENTARY

C3.11.5.7

The following shall supplement the first paragraph of AC3.11.5.7.

The earth pressure diagrams in Figures A3.11.5.7.1-1 and A3.11.5.7.2b-1 are primarily intended for use in homogeneous soils. They should not be used indiscriminately in stratified or relatively non-homogeneous soil layers; engineering judgment must be used in these cases.

When anchors, especially those near the top of the wall, are tensioned to loads in excess of those estimated using the apparent pressure diagrams, it is possible that the wall could be displaced back into the soil mass, resulting in undesirable deflections or a passive failure of the retained soil. It is important to remember that anchored walls are flexible and that they derive their satisfactory performance from a match between the soil pressure and the wall-anchor loads.

C3.11.5.7.1P

Anchored walls are typically constructed with free-draining material placed immediately behind the lagging, and therefore geostatic water pressure on the wall would not be of concern. However, there may be conditions of a permanent water table behind the wall where geostatic water pressure needs to be considered.

C3.11.5.7.2P

Cohesive soils with a stability number $N_S \leq 4$ are to be considered to be stiff to hard in consistency. Cohesive soils with a stability number $N_S > 4$ are to be considered soft to medium-stiff in consistency.

C3.11.5.7.2a

The following shall supplement AC3.11.5.7.2:

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should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall. For conditions where there is available free water, determine geostatic water pressure on the wall using the maximum expected water table differential between excavation interior and exterior, based on borings or other information.

Alternatively, in fissured clays the apparent earth pressure diagram may be based upon previous successful experience with excavations constructed in similar soils. This is because earth pressures in these soils are most influenced by degree of fissuring or jointing in the clay and the potential reduction in strength with time, not necessarily the shear strength of the intact clay.

3.11.5.7.2b *Soft to Medium Stiff*

The following shall replace A3.11.5.7.2b:

The apparent earth pressure distribution for temporary and permanent anchored walls constructed from the top down and supporting soft to medium stiff cohesive soils may be determined using Figure A3.11.5.7.2b-1. Soft to medium stiff cohesive soils are those with a stability number $N_s > 4$.

Calculate the maximum pressure ordinate of the diagram p as:

$$\rho = 1.0 k_a \gamma_s H \quad (3.11.5.7.2b-1)$$

where:

$$k_a = 0.22 \text{ for } 4 \leq N_s < 5.14$$

or

$$k_a = 1 - m \frac{4S_u}{\gamma_s H} \quad (3.11.5.7.2b-2)$$

for $N_s \geq 5.14$, and using $m = 0.4$

Additionally, if $N_s \geq 6$ and the excavation is underlain by soft clay, calculate k_a by Equation 3.11.5.7.2b-2 and Equation 3.11.5.7.2b-3 below, and use the larger of the two k_a values in Equation 3.11.5.7.2b-1 to calculate the maximum pressure ordinate.

$$k_a = 1 - \frac{4S_u}{\gamma_s H} + \frac{2\sqrt{2}d}{H} \left\{ 1 + \frac{\Delta H}{H} a - \frac{S_{ub}}{\gamma_s H} b \right\} \quad (3.11.5.7.2b-3)$$

where:

$$a = 1 + \frac{H + \Delta H}{(2-x)\sqrt{2}d} \quad (\text{dim})$$

COMMENTARY

There may be conditions of a permanent water table behind the wall where geostatic water pressure needs to be considered.

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COMMENTARY

$$b = 5.14 + \frac{2S_u \Delta H}{\sqrt{2} S_{ub} d} \quad (\text{dim})$$

S_u = undrained shear strength of retained soil (ksf)

S_{ub} = undrained shear strength of soil providing bearing resistance (ksf)

d = depth of the potential base failure surface below the base of excavation (ft.)

γ_s = total unit weight of retained soil (kcf)

ΔH = depth of unloading at ground surface, if any (ft.)

x = length of unloading at top of anchored wall excavation, if any (ft.)

The value of d is taken as the thickness of soft to medium stiff cohesive soil below the excavation base up to a maximum value of $B_c \sqrt{2}$, where B_c is the excavation width.

In any case, surcharge pressures, if applicable, should be added explicitly to the diagrams to evaluate the total lateral load acting on the wall. For conditions where there is available free water, determine geostatic water pressure on the wall using the maximum expected water table differential between excavation interior and exterior, based on borings or other information.

There may be conditions of a permanent water table behind the wall where geostatic water pressure needs to be considered.

3.11.5.8 Lateral Earth Pressures for Mechanically Stabilized Earth Walls

3.11.5.8.1 General

The following shall replace the definition of k_a in A3.11.5.8.1:

k_a = active earth pressure coefficient specified herein

The following shall supplement A3.11.5.8.1:

Lateral earth pressure coefficients for MSE walls may be determined as follows:

- for a horizontal or sloping backfill surface, as shown in Figures A3.11.5.8.1-1 and A3.11.5.8.1-2, active earth pressure coefficient, k_a , in determining safety against soil failure may be taken as:

$$k_a = \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi_f}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi_f}} \quad (3.11.5.8.1-2)$$

where:

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COMMENTARY

β = slope of backfill behind wall (degrees)

ϕ_f = internal friction angle of backfill soil (degrees)

- for a broken back backfill surface, the active earth pressure coefficient, k_a , for evaluation of safety against soil failure may be taken as:

$$k_a = \cos B \frac{\cos B - \sqrt{\cos^2 B - \cos^2 \phi_f}}{\cos B + \sqrt{\cos^2 B - \cos^2 \phi_f}} \quad (3.11.5.8.1-3)$$

where:

B = notional slope of backfill behind wall as shown in Figure A3.11.5.8.1-3 (degrees)

ϕ_f = internal friction angle (degrees)

- active earth pressure coefficient, k_a , for determining safety against structural failure:

$$k_a = \tan^2 \left(45^\circ - \frac{\phi_f}{2} \right) \quad (3.11.5.8.1-4)$$

3.11.6 Surcharge Loads: ES and LS

3.11.6.4 Live Load Surcharge (LS)

The following shall replace Table A3.11.6.4-1.

Table 3.11.6.4-1 - Equivalent Height of Soil for Vehicular Loading – Abutment

Wall Height (ft)	H_{eg} (ft)
≤ 5.0	4.0
≥ 10.0	3.0

The following shall supplement A3.11.6.4.

The minimum design surcharge values for abutments in Table D3.11.6.4-1 are intended to account for normal traffic live loads and do not address the effects of backfill compaction. Refer to A3.11.2 to determine the effects of backfill compaction. For retaining walls, use Table D3.11.6.4-2.

C3.11.6.4

Delete the third paragraph of AC3.11.6.4.

The following shall supplement AC3.11.6.4.

In the development of this specification, the Department had a comparison made between their past abutment and retaining wall service load design method and the LRFD method. With minor modifications contained in this specification, the LRFD method gave similar results to the Department's past design method with one exception. For walls less than 5 ft. in height on poor soils, the LRFD method may require base width significantly larger than past designs. Since the Department has not experienced problems with short headwalls for pipe culverts, the Standard Drawings may be used for headwalls for pipe culverts.

SPECIFICATION

Table 3.11.6.4-2 - Equivalent Height of Soil (h_{eq}) for Vehicular Loading - Retaining Walls

Wall Height (ft)	Distance from back face of wall to the wheel line	
	0.0 ft	1 ft
≤ 5.0	5.0	3.0
10.0	3.5	3.0
≥ 13.0	3.0	3.0

For box culverts, use 3.0 ft. where live load effects are considered.

3.12 FORCE EFFECTS DUE TO SUPERIMPOSED DEFORMATIONS: TU, TG, SH, CR, SE, PS

3.12.2 Uniform Temperature

3.12.2.1 Temperature Range for Procedure A

The following shall replace A3.12.2.1.

Provision shall be made for forces and movements resulting from variations in temperature. The range of temperature with respect to the normal erection temperature of 68° F shall be as given in Table D3.12.2.1-1.

For the design of integral abutments, the temperature range given for bearings shall be used.

COMMENTARY

In Table D3.11.6.4-2, the distance from back face of wall to edge of traveled way of 0 ft. corresponds to placement of a point wheel load 2 ft. from the back face of the wall. For the case of the uniformly distributed lane load, the 0 ft. distance corresponds to the edge of the 10 ft. wide traffic lane.

C3.12.2.1

The following shall supplement AC3.12.2.1.

The increase in the concrete structure temperature range is based on a Departmental study conducted in District 3-0. It was determined that the fixity at the connections of continuous spans produces a frame-type action that induces additional forces.

Table 3.12.2.1-1 – Procedure A Temperature Ranges

Superstructure Material	Temperature Rise	Temperature Fall	Range
Steel or Aluminum Structures	42° F	78° F	-10° F to 110° F
Concrete Structures	32° F	58° F	10° F to 100° F
Wood Structures	7° F	14° F	54° F to 75° F
Bearing Design Temp. Range	Neoprene	Other	---
Prestressed Concrete Structures	80° F	116° F	---
Steel or Aluminum Structures	100° F	156° F	---

3.12.3 Temperature Gradient

The following shall supplement A3.12.3.

The load factor for temperature gradient shall be taken as zero for those bridges which can be analyzed by the approximate methods given in A4.6.2 and D4.6.2, and are of Type a, b (only precast P/S concrete box girders), e, f, g, h, j, k and l as given in Table A4.6.2.2.1-1.

For Pennsylvania bridges other than those listed above,

C3.12.3

The following shall supplement AC3.12.3.

Pennsylvania has not experienced any temperature gradient-related problems in their typical multi-girder bridges. Therefore, as suggested in AC3.12.3, the Department's experience with typical multi-girder bridges has led them to exclude the temperature gradient load condition for these types of bridges.

SPECIFICATION

the Zone 3 data shall be used as given in Table A3.12.3-1.

3.12.8P Minimum Temperature Force for Fixed Substructures

When neoprene bearings are used, the fixed substructure unit(s) shall consider a thermal force equal to the largest thermal force from the largest expansion bearing substructure unit or utilize the results of an equilibrium analysis, whichever is larger.

3.12.9P Temporary Support Settlement for Curved or Skewed Bridges during Construction

When a temporary falsework is used, an analysis should be performed to check its settlement effects on member response during construction. As a minimum, the following scenarios should be considered for the analysis:

- Settlement of single and multiple temporary supports
- A minimum settlement of one thousandth of the span length should be used

3.13 FRICTION FORCES: FR

The following shall supplement A3.13.

Friction force acts parallel to the direction of movement and is assumed to act at the bearing elevation at each expansion bearing, with due consideration given to the reactions that must develop at the fixed bearings to satisfy equilibrium. See A14.6.3.1 for horizontal forces.

Consideration of frozen expansion bearings and variation of friction is provided assuming the largest pier or abutment DL reaction times the applicable friction coefficient acts at the fixed pier or utilize the results of an equilibrium analysis, whichever is larger.

For design purposes, the coefficient of friction between the PTFE and stainless steel sliding surfaces of pot bearings may be taken as 0.04 based on dimpled and lubricated PTFE.

3.14 VESSEL COLLISION: CV**3.14.1 General**

The following shall supplement A3.14.1.

The vessel collision provisions provided in A3.14 and D3.14 shall only be used in the substructure design of bridges which cross a navigable waterway. The Department defines a navigable waterway as those waterways which:

- presently support commercial barge and/or ship traffic
- have supported commercial barge and/or ship traffic

COMMENTARY

C3.12.8P

This provision insures that fixed substructures are designed for a minimal thermal force even if an equilibrium analysis indicates no thermal forces are present. This is similar to the forces applied to steel bearings considering frozen bearings.

C3.12.9P

The information in this Article is based on Penn State Research Report titled Guideline for Analyzing Curved and Skewed Bridges and Designing them for Construction, August 15, 2010.

C3.13

The following shall replace AC3.13.

Low and high friction coefficients may be obtained from standard textbooks. If so warranted and approved by the Chief Bridge Engineer, the values may be determined by physical tests, especially if the surfaces are expected to be roughened in service.

When a force is transmitted from the superstructure to the substructure through a sliding bearing, the force applied to the substructure is considered a frictional force. However, forces transmitted, via a non-sliding bearing such as an elastomeric bearing, are factored by the appropriate load factor for the driving effect, such as TU.

Pot bearings, as shown on Standard Drawing BD-613M, were developed using a coefficient of friction between the PTFE and stainless steel equal to 0.04.

C3.14.1

The following shall supplement AC3.14.1.

For the vast majority of bridges over waterways in Pennsylvania, the vessel collision provisions will not be applicable.

The vessel collision provisions will most likely be applicable for bridges over the following waterways:

- lower portions of Delaware River

SPECIFICATION

within the past 20 years

- There is some reason to believe that the waterway will support commercial barge and/or ship traffic in the future

COMMENTARY

- lower portions of Schuylkill River
- lower portions of Allegheny River
- lower portions of Monongahela River
- Ohio River

The following structures may be used as reference projects for vessel collision design requirements:

- S-22234 – Ford City
- S-27067 – Point Marion
- S-30378 – Charleroi-Monessen
- S-30980 – Masontown

3.14.2 Owner's Responsibility

The following shall replace A3.14.2.

When the vessel collision provisions are applicable according to D3.14.1, the designer must submit at the Type, Size and Location stage the following information for review by the Department:

- vessel traffic density in the waterway
- design velocity of vessels for the bridge
- suggested degree of damage that the bridge components, including protective systems are allowed to sustain

3.14.15 Protection of Substructures**C3.14.15**

The following shall supplement AC3.14.15.

Any testing for protection systems of substructures must be approved by the Chief Bridge Engineer.

3.17P FORCE TRANSFER TO SUBSTRUCTURE**3.17.1P Longitudinal Force****3.17.1.2P Force Transfer to Substructure**

Longitudinal forces, except friction (see D3.13), shall be carried only by fixed bearings.

3.17.1.3P Effective Length for Superstructure Forces

Longitudinal forces transmitted to the substructure from the superstructure shall be calculated using the center-to-center bearing length of superstructure restrained by fixed bearings. In the case of consecutively fixed piers, forces to the substructure shall be determined with due consideration to the relative stiffness of the piers.

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COMMENTARY

3.17.1.4P Force Resolution to Substructure

Longitudinal forces from the superstructure shall be directly applied at the bearings and shall be resolved in the directions perpendicular and parallel to the substructure, as shown in Figure 3.17.1.4P-1. For frame analysis of the substructure, an equivalent parallel component shall be used, as shown in Figure 3.17.1.4P-2.

For structures on a sloping grade with an inclined bearing plate, the reaction component parallel to the grade (longitudinal force) shall be considered.

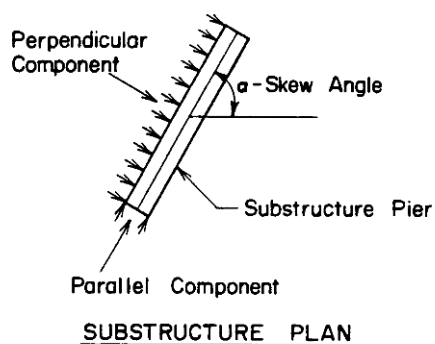
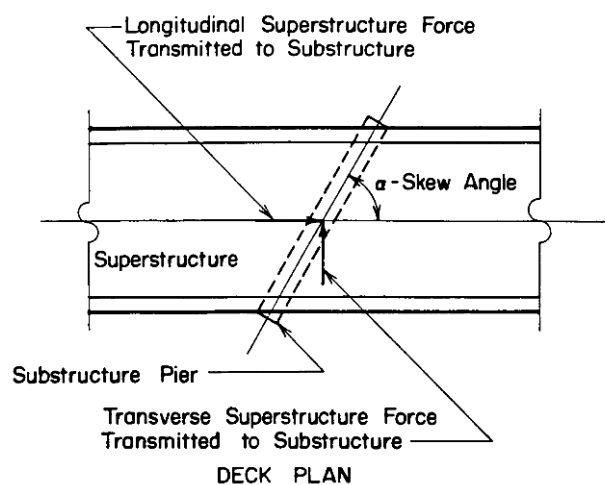


Figure 3.17.1.4P-1 - Force Resolution to Substructure

SPECIFICATION

COMMENTARY

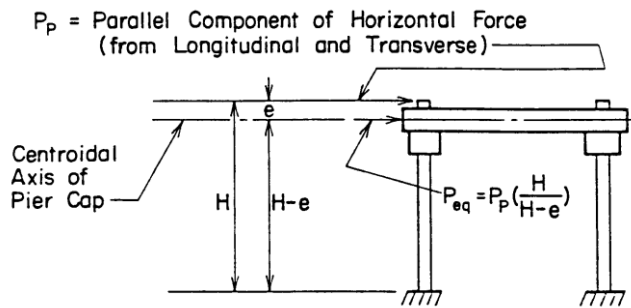


Figure 3.17.1.4P-2 - Equivalent Force for Frame Analysis of Substructure

3.17.2P Transverse Force

3.17.2.1P Force Transfer to Substructure

The transverse forces applied to the superstructure must be resisted by the bearings.

3.17.2.2P Effective Lengths for Superstructure Forces

Unless a more rational method of analysis is used, transverse forces acting on a superstructure shall be transmitted to the bearings using the following span lengths:

Continuous Spans	Piers	Average of the two adjacent spans
	Abutments	One-half of the end span
Simple Spans		One-half of the span

3.17.2.3P Force Resolution to Substructure

Transverse forces from the superstructure shall be resolved in the directions perpendicular and parallel to the substructure, as shown in Figure D3.17.1.4P-1.

3.17.2.4P Determination of Bearing Reactions

The effect of the transverse force applied at the elevation specified for that force shall be taken into account in determining the vertical reactions at the bearings (see Figure 3.17.2.4P-1).

SPECIFICATION

COMMENTARY

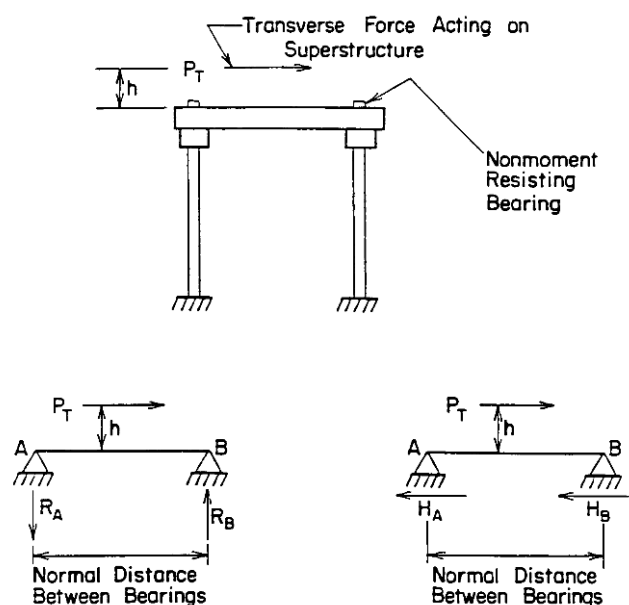


Figure 3.17.2.4P-1 - Bearing Reactions

3.18P REFERENCES

See A3.16 for AASHTO References.

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

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4.1 SCOPE

C4.1

The following shall replace the last paragraph of A4.1.

Bridge structures shall be analyzed elastically, except as noted herein. An inelastic analysis of bridge structures may only be used for Extreme Event Limit States with the approval of the Chief Bridge Engineer.

The following shall supplement AC4.1.

For construction situations of curved or skewed bridges, where second-order effects are significant, the engineer should consider a geometric nonlinear analysis to more accurately determine the second-order effects.

4.2 DEFINITIONS

The following shall supplement A4.2.

Automatic Mesh Generator - Program or subprogram which creates the layout (arrangement of nodes and elements) of a model for the user if certain basic information is provided.

Influence Surface - Curved surface on which the ordinate is the value of the function (shear, moment, reaction, etc.) when a unit load is placed at the ordinate for a member location (centerline of a girder, support, etc.).

Influence Surface Loader - Computer program or portion of a computer program which calculates and maximizes the value of a function (shear, moment, reaction, deflection, etc.) by using the influence surface for that function.

Line Girder Analysis - Analysis of a bridge in which each girder is removed and analyzed as a single non-interacting element.

Loading Algorithm - Methodology used by the influence surface loader to calculate the moments, shear, etc.

Refined Analysis - Analysis according to A4.6.3 and D4.6.3.

Three-Dimensional Finite Element Analysis - Analysis in which a three-dimensional continuum is modeled as an assemblage of discrete elements in three-dimensional space.

Warping Torsion - Nonuniform torsion resulting in warping of the cross-section.

The following shall replace the definition of skew angle in A4.2

Skew Angle - Angular measurement between the bridge centerline (or tangent thereto) and a line parallel to the support; a 90° skew angle defining a right bridge (See PP3.2.2).

4.4 ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

The following shall supplement A4.4.

The designer shall also follow the requirements in PP1.4 in regards to computer programs.

Any computer program for the "3D or refined" analysis of girder bridges which has not been reviewed by the Department shall be submitted to, and approved by, the Chief Bridge Engineer prior to its use. A sample bridge(s) selected by the Department is to be modeled with the program so that the Department can make comparisons between its reviewed programs and the proposed program. Computer programs for the analysis of girder bridges approved for use on LRFD design projects are listed in Appendix J. Only the version of a program listed in Appendix J has been tested and approved. If any changes

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and/or modifications have been made to a program since its approval date, then re-approval of the program is required. The approval of these programs is subject to the following conditions and limitations:

1. While certain software packages provide design optimization and/or code compliance checks, these aspects were not included in the review and approval process. Acceptance has been based solely upon the review of generalized design forces (moments, shears, reactions, etc.), as calculated by the software.
2. Acceptance of a software package by the Department does not affect the responsibility of the user for the proper application of the software and interpretation of its results. The acceptance of a software package does not constitute an endorsement nor does it relieve the vendor and the designer from their responsibility for accurate, technically correct and sound engineering results and services to the Department.
3. The Department's acceptance does not constitute any form of implied warranty, including warranty of merchantability and fitness for a particular purpose. The Commonwealth makes no warranty or representation, either expressed or implied, with respect to this software or accompanying documentation, including their quality performance, merchantability, or fitness for a particular purpose. In addition, the Commonwealth will not be liable for any direct, indirect, special, incidental, or consequential damages arising out of the use, inability to use, or any defect in the software or any accompanying documentation.

4.5 MATHEMATICAL MODELING**4.5.1 General**

The following shall replace the second paragraph of A4.5.1.

Barriers shall not be considered in the calculation of the structural stiffness nor structural resistance of a structure.

The following shall supplement A4.5.1.

Centerline distances shall be used in the analysis of continuous frames, such as boxes, arches and pier bents.

4.5.2 Structural Material Behavior**4.5.2.2 Elastic Behavior**

The following shall supplement A4.5.2.2.

For simple and continuous spans, composite stiffness shall be used if a concrete deck is used.

C 4.5.1

The following shall replace the first paragraph of AC 4.5.1.

Strength, service and fatigue limit states shall be analyzed as fully elastic, except as noted herein. An inelastic analysis of bridge structures may only be used for Extreme Event limit states with the approval of the Chief Bridge Engineer.

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4.5.2.3 Inelastic Behavior

The following shall replace the first sentence of the second paragraph of A4.5.2.3.

The inelastic model shall be based either upon the results of physical tests or upon a representation of load deformation behavior which is validated by tests, but either method must be approved by the Chief Bridge Engineer.

4.6 STATIC ANALYSIS**4.6.1 Influence of Plan Geometry****4.6.1.2 Structures Curved in Plan***4.6.1.2.1 General*

The following shall supplement A4.6.1.2.1.

Bridges which have kinked girders shall use the provisions of A4.6.1.2.2, A4.6.1.2.3 and A4.6.1.2.4 to determine if they are to be considered curved.

For the design of horizontally curved steel girder highway bridges, a load and resistance factor design (LRFD) is required. The force effects (i.e., moments, shear, reactions, etc.) for the curved bridge shall be determined using a refined method of analysis.

C4.6.1.2.1

The following shall supplement AC4.6.1.2.1

The selected refined method of analysis for a structure curved in plan must provide an accurate prediction of behavior, both during construction and while in-service. While the method of analysis that is selected is at the discretion of the designer, a superstructure modeling technique that represents the girder webs and concrete deck using shell elements and other major superstructure components using beam elements provides an acceptable compromise between reduced computation times provided by grillage analogy models and increased accuracy provided by more sophisticated three-dimensional finite element models.

4.6.2 Approximate Methods of Analysis**4.6.2.1 Decks***4.6.2.1.3 Width of Equivalent Interior Strips*

In Table A4.6.2.1.3-1 replace the entry for wood planks as follows:

The width of a primary wood plank strip spanning parallel to traffic shall be taken as 20 in.. The width of a primary wood plank strip spanning perpendicular to traffic shall be taken as the plank width, but not less than 10 in..

*4.6.2.1.4c Transverse Edges**C4.6.2.1.4c*

Delete AC4.6.2.1.4c

*4.6.2.1.6 Calculation of Force Effects**C4.6.2.1.6*

Delete the first paragraph of AC4.6.2.1.6.

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4.6.2.1.8 Live Load Force Effects for Fully and Partially Filled Grids and for Unfilled Grid Decks Composite with Reinforced Concrete Slabs

The following shall replace A4.6.2.1.8.
Design in accordance with Tables per BD-604M.
The stiffness ratio, D, shall be taken as:

- for fully filled grids with at least 1 1/2 in. monolithic overfill 2.0
- for all other fully filled grids 2.5
- for partially filled grids with at least 1 1/2 in. monolithic overfill 8.0
- for all other partially filled grid 10.0

When approved by the Chief Bridge Engineer, the stiffness ratio, D, determined from test results may be used.

4.6.2.1.9 Inelastic Analysis

The following shall replace A4.6.2.1.9

The inelastic finite element analysis or yield line analysis are not permitted unless specifically approved by the Chief Bridge Engineer. If approved, this type of analysis is to be only used for Extreme Event Limit State.

4.6.2.2 Beam-Slab Bridges**4.6.2.2.1 Application**

The following shall replace the fourth paragraph of A4.6.2.2.1.

For any variables exceeding the range of applicability, as specified in A4.6.2.2 and D4.6.2.2, the Chief Bridge Engineer must approve the method for determining the distribution factors.

The following shall supplement A4.6.2.2.1.

The articles in this section which provide approximate distribution factors are not applicable for bridges which are considered curved as defined in A4.6.1.2.2, A4.6.1.2.3 and A4.6.1.2.4 and D4.6.1.2.1. For curved bridges, a refined method of analysis, as defined in A4.6.3 and D4.6.3, is required.

Additional requirements for skewed structures must be considered as follows:

- Apply the skew adjustment factors as given in A4.6.2.2.3c and D4.6.2.2.3c on all skewed structures as a minimum.
- Steel structures with a skew angle less than 70° require an additional check against uplift at the acute corners.

C4.6.2.2.1

The following shall replace the fifth sentence of the thirteenth paragraph of AC4.6.2.2.1.

The use of transverse mild steel rods secured by nuts, or similar unstressed dowels should not be considered sufficient to achieve full transverse flexural continuity unless demonstrated by test or experience and approved by the Chief Bridge Engineer.

The following shall supplement AC4.6.2.2.1.

AASHTO provides consideration of skew angle by way of moment and shear correction factors. PennDOT agrees with the application of the shear correction factors. PennDOT has decided not to take advantage of the reduction in load distribution factors for moment. However, these factors do not adequately address problems due to out-of-plane rotations, uplift, or cross-frame forces. The provisions in this section are meant to be applied to account for these items. Note that uplift on concrete structures is not

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- Concrete structures with a skew angle less than 45° require an additional check against uplift at the acute corners.
- The design of bearings for bridges with skew angles less than 70° require consideration of out-of-plane rotations.
- Steel structures with skew angles less than 70° require a special cross-frame design and the cross-frame members must be considered as main load carrying members.

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considered as critical as that on steel structures.

During routine bridge inspections, the Department has found many occurrences of buckled cross-frame members and poor bearing performance on skewed structures. The Department has found from refined analyses that cross-frames in skewed structures are potentially subjected to higher force levels than cross-frames in normal (90°) structures. This does not mandate a 3-D analysis, but does mean a special analysis of the cross-frame must be provided in order to account for the differential deflections which occur across a cross-frame. This analysis should accurately account for cross frame member geometry and stiffness. Should a grillage analogy model be used, accurate representation of cross frame stiffness should be established via special analysis of representative frames. Should a more sophisticated 3-D analysis be used, models can be constructed following the technique recommended in DC4.6.1.2.1 for structures curved in plan.

A crude or approximate uplift check can be made using the second term of the adjustment factor as an estimate of negative live load reaction potential. Comparing the negative reaction with the dead load reaction will provide an estimate of the potential for uplift. The engineer must use engineering judgement regarding the applicability of this method.

An acute corner is defined as the corner of the structure where the angle formed by intersection of edge of the deck and the centerline of bearings is less than 90°.

Proper consideration of out-of-plane rotations during the bearing design is also required. Normally out of plane rotations will require multi-rotational bearings.

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The following shall supplement A4.6.2.2.1.

Table D4.6.2.2.1-1 describes how the term L (length) shall be determined for use in the live load distribution factor equations given in A4.6.2.2.2, A4.6.2.2.3, D4.6.2.2.2 and D4.6.2.2.3.

The following shall replace Table C4.6.2.2.1-1 in AC4.6.2.2.1.

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The following shall supplement AC4.6.2.2.1.

The method incorporated in this manual for determining L seems appropriate for the level of sophistication in the live load distribution factor equations. As additional knowledge is gained on this subject, this method for determining L may be modified.

Table 4.6.2.2.1-1 - L for Use in Live Load Distribution Factor Equations

CONDITION	FORCE EFFECT	L (ft.)
A	Positive Moment	The length of the span for which moment is being calculated.
B	Negative Moment - End spans of continuous spans, from end to point of dead load contraflexure	The length of the span for which moment is being calculated.
C	Negative Moment - Near interior supports of continuous spans, from point of dead load contraflexure to point of dead load contraflexure	The average length of the two adjacent spans.
D	Negative Moment - Interior spans of continuous spans, from point of dead load contraflexure to point of dead load contraflexure	The length of the span for which moment is being calculated.
E	Shear	The length of the span for which shear is being calculated.
F	Exterior Reaction	The length of the exterior span.
G	Interior Reaction of Continuous Span	The average length of the two adjacent spans.

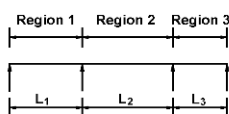
Figure 4.6.2.2.1P-1 provides a graphical representation of the information given in Table D4.6.2.2.1-1.

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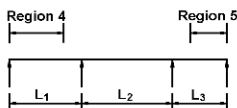
L for use in Live Load Distribution Factor Equations

Condition A



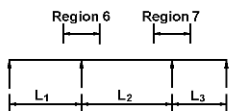
For Positive Moment Distribution Factors in Regions 1, 2 & 3, L_1 , L_2 & L_3 would be used in the Distribution Factor Equations respectively.

Condition B



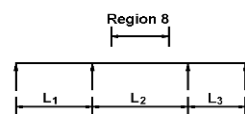
For Negative Moment Distribution Factors in Region 4 & 5, L_1 & L_3 would be used in the Distribution Factor Equations respectively.

Condition C



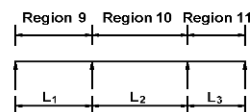
For Negative Moment Distribution Factors in Regions 6 & 7, $\frac{L_1 + L_2}{2}$ & $\frac{L_2 + L_3}{2}$ would be used in the Distribution Factor Equations respectively.

Condition D



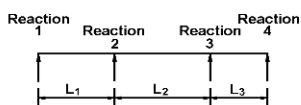
For the Negative Moment Distribution Factor in Region 8, L_2 would be used in the Distribution Factor Equations.

Condition E



For Shear Distribution Factors in Regions 9, 10 & 11, L_1 , L_2 & L_3 would be used in the Distribution Factor Equations respectively.

Condition F & G



For Reaction Distribution Factors for Reactions 1, 2, 3 & 4, L_1 , $\frac{L_1 + L_2}{2}$, $\frac{L_2 + L_3}{2}$ & L_3 would be used in the Distribution Factor Equations respectively.

Figure 4.6.2.2.1P-1 - L for use in Live Load Distribution Factor Equations

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In the rare occasion when the continuous span arrangement is such that an interior span does not have any positive dead load moment (i.e., no dead load points of contraflexure), the region of negative moment near the interior supports would be increased to the centerline of the span, and the L used in determining the live load distribution factors would be the average of the two adjacent spans.

4.6.2.2.2 Distribution Factor Method for Moment and Shear

4.6.2.2.2a Interior Beams with Wood Decks

The following shall supplement A4.6.2.2.2a.

The distribution factors given in Table A4.6.2.2.2a-1 for Glued Laminated Panels on Glued Laminated Stringers are applicable for panels with a 6 in. minimum nominal thickness.

4.6.2.2.2b Interior Beams with Concrete Decks

The following shall supplement A4.6.2.2.2b.

For preliminary design, the term $K_g/(12Lt^3)$ in Table D4.6.2.2.2b-1 shall be taken as 1.0 for non-composite beams.

The following shall replace Table A4.6.2.2.2b-1.

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C4.6.2.2.2b

The following shall supplement AC4.6.2.2.2b.

In Table A4.6.2.2.2b-1, in the Category "Concrete Beams used in Multi-Beam Decks", the cross-section, Type g (from Table A4.6.2.2.1-1), with option "if sufficiently connected to act as a unit" has been removed from Table D4.6.2.2.2b-1. This option has been removed because it has been difficult to provide enough post-tensioning for the composite box beams to act as a unit.

Table 4.6.2.2b-1 – Distribution of Live Loads Per Lane for Moment in Interior Beams

Type of Beams	Applicable Cross-Section from Table A4.6.2.2.1-1	Distribution Factors	Range of Applicability
Wood Deck on Wood or Steel Beams	a, l	See Table A4.6.2.2a-1	
Concrete Deck on Wood Beams	l	One Design Lane Loaded: $S/12$ Two or More Design Lanes Loaded: $S/10$	$S \leq 6.0'$
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections or NEXT Beams	a, e, k	One Design Lane Loaded: $0.06 + \left(\frac{S}{9.6}\right)^{1.0} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$	$3.5' \leq S \leq 16.0'$ $4 \frac{1}{2}" \leq t_s \leq 12"$ $20' \leq L \leq 500'$ If $L > 240'$, use $L = 240'$ $10,000 \text{ in}^4 \leq K_g \leq 7,600,000 \text{ in}^4$ $N_b \geq 4$
	i, j If sufficiently connected to act as a unit	Two or More Design Lanes Loaded: $0.075 + \left(\frac{S}{11}\right)^{1.0} \left(\frac{S}{L}\right)^{0.08} \left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$	
		Use lesser of the above or Lever Rule	$N_b = 3$
Cast-in-Place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75 + \frac{S}{3.6}\right) \left(\frac{1}{L}\right)^{0.35} \left(\frac{1}{N_c}\right)^{0.45} \geq \frac{S}{21}$ Two or More Design Lanes Loaded: $\left(\frac{13}{N_c}\right)^{0.3} \left(\frac{S}{5.8}\right) \left(\frac{1}{L}\right)^{0.25}$	$7.0' \leq S \leq 13.0'$ $60' \leq L \leq 500'$ If $L > 240'$, use $L = 240'$ $N_c \geq 3$ If $N_c > 8$ use $N_c = 8$ For two or more design lanes loaded If $L > 1400/N_c$, use $L = 1400/N_c$
Concrete Deck on Concrete Spread Box Beams	b, c	One Design Lane Loaded: $\left(\frac{S}{3.0}\right)^{0.35} \left(\frac{Sd}{12L^2}\right)^{0.25}$ Two or More Design Lanes Loaded: $\left(\frac{S}{6.3}\right)^{0.6} \left(\frac{Sd}{12L^2}\right)^{0.125}$	$6.0' \leq S \leq 18.0'$ $20' \leq L \leq 500'$ If $L > 140'$, use $L = 140'$ $17" \leq d \leq 66"$ $N_b \geq 3$
		Use Lever Rule	$S \geq 18.0'$

Table 4.6.2.2b-1 – Distribution of Live Loads Per Lane for Moment in Interior Beams (Continued)

Type of Beams	Applicable Cross-Section from Table A4.6.2.2.1-1	Distribution Factors	Range of Applicability												
Concrete Beams used in Multi-Beam Decks	f	One Design Lane Loaded: $k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$ where: $k = 2.5(N_b)^{-0.2} \geq 1.5$	$35'' \leq b \leq 60''$ $20' \leq L \leq 500'$ If $L > 120'$, use $L = 120'$ $5 \leq N_b \leq 20$												
	g	Two or More Design Lanes Loaded: $k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}\left(\frac{2.4}{L^{0.15}}\right)^{\frac{N_b}{15}}$	$35'' \leq b \leq 60''$ $20' \leq L \leq 500'$ $5 \leq N_b \leq 20$ If $N_b > 12$, use $N_b = 12$												
	h	Regardless of Number of Loaded Lanes: S/D where: $C=K(W/L)$													
	g, i, j if connected only enough to prevent relative vertical displacement at the interface	$D = 11.5 - N_L + 1.4N_L(1 - 0.2C)^2$ When $C \leq 5$ $D = 11.5 - N_L$ when $C > 5$ $K = \sqrt{\frac{(1 + \mu)l}{J}}$ for preliminary design, the following values of K may be used: <table><tr><td><u>Beam Type</u></td><td><u>K</u></td></tr><tr><td>Non-voided rectangular beams</td><td>0.7</td></tr><tr><td>Rectangular beams with circular voids</td><td>0.8</td></tr><tr><td>Box section beams</td><td>1.0</td></tr><tr><td>Channel beams</td><td>2.2</td></tr><tr><td>T-beam</td><td>2.0</td></tr><tr><td>Double T-beam</td><td>2.0</td></tr></table>		<u>Beam Type</u>	<u>K</u>	Non-voided rectangular beams	0.7	Rectangular beams with circular voids	0.8	Box section beams	1.0	Channel beams	2.2	T-beam	2.0
<u>Beam Type</u>	<u>K</u>														
Non-voided rectangular beams	0.7														
Rectangular beams with circular voids	0.8														
Box section beams	1.0														
Channel beams	2.2														
T-beam	2.0														
Double T-beam	2.0														
Steel Grids on Steel Beams	a	One Design Lane Loaded: $S/7.5'$ If $t_g < 4''$ $S/10.0'$ If $t_g \geq 4''$	$S \leq 6.0'$												
		Two or More Design Lanes Loaded: $S/8.0'$ If $t_g < 4''$ $S/10.0'$ If $t_g \geq 4''$	$S \leq 6.0'$												
Concrete deck on Multiple Steel Box Girders	b, c	Regardless of Number of Loaded Lanes: $0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L}$	$0.5 \leq \frac{N_L}{N_b} \leq 1.5$												

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*4.6.2.2.2d Exterior Beams**C4.6.2.2.2d*

The following shall supplement AC4.6.2.2.2d.

The value of d_e is to be computed using the midpoint of the exterior web.

*4.6.2.2.2e Skewed Bridges**C4.6.2.2.2e*

Delete A4.6.2.2.2e.

The following shall replace AC4.6.2.2.2e.

PennDOT has decided not to take advantage of the reduction in load distribution factors for moment in longitudinal beams on skewed supports.

*4.6.2.2.3 Distribution Factor Method for Shear**4.6.2.2.3a Interior Beams*

The following shall replace the second sentence of Paragraph 1 in A4.6.2.2.3a.

For interior beams not listed in Table D4.6.2.2.3a-1, lateral distribution of axle load shall be determined by lever rule.

The following shall replace Table A4.6.2.2.3a-1.

Table 4.6.2.2.3a-1 – Distribution of Live Loads Per Lane for Shear in Interior Beams

Type of Superstructure	Applicable Cross-Section from Table A4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Wood Deck on Wood or Steel Beams	See Table A4.6.2.2.2a-1			
Concrete Deck on Wood Beams	l	Lever Rule	Lever Rule	N/A
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections or NEXT Beams	a, e, k	$0.36 + \frac{S}{25}$	$0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2$	$3.5' \leq S \leq 16.0'$ $20' \leq L \leq 500'$ If $L > 240'$, use $L = 240'$ $4 \frac{1}{2}" \leq t_s \leq 12"$ $N_b \geq 4$
	i, j If sufficiently connected to act as a unit			
Cast-in-Place Concrete Multicell Box	d	$\left(\frac{S}{9.5}\right)^{0.6} \left(\frac{d}{12L}\right)^{0.1}$	$\left(\frac{S}{7.3}\right)^{0.9} \left(\frac{d}{12L}\right)^{0.1}$	$6.0' \leq S \leq 13.0'$ $20' \leq L \leq 500'$ $35" \leq d \leq 110"$ $N_c \geq 3$
Concrete Deck on Concrete Spread Box Beams	b, c	$\left(\frac{S}{10}\right)^{0.6} \left(\frac{d}{12L}\right)^{0.1}$	$\left(\frac{S}{7.4}\right)^{0.8} \left(\frac{d}{12L}\right)^{0.1}$	$6.0' \leq S \leq 18.0'$ $20' \leq L \leq 500'$ If $L > 140'$, use $L = 140'$ $17" \leq d \leq 66"$ $N_b \geq 3$
		Lever Rule	Lever Rule	$S > 18.0'$
Concrete Box Beams Used in Multi-Beam Decks	f, g	$\left(\frac{b}{130L}\right)^{0.15} \left(\frac{I}{J}\right)^{0.05}$	$\left(\frac{b}{156}\right)^{0.4} \left(\frac{b}{12L}\right)^{0.1} \left(\frac{I}{J}\right)^{0.05}$	$35" \leq b \leq 60"$ $20' \leq L \leq 500'$ If $L > 120'$, use $L = 120'$ $5 \leq N_b \leq 20$ $16,500 \text{ in}^4 \leq J \leq 900,000 \text{ in}^4$ $5,000 \text{ in}^4 \leq I \leq 900,000 \text{ in}^4$
Concrete Beams Other Than Box Beams Used in Multi-Beam Decks	h	Lever Rule	Lever Rule	N/A
	i, j if connected only enough to prevent relative vertical displacement at the interface			
Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A
Concrete Deck on Multiple Steel Box Beams	b, c	As specified in Table D4.6.2.2.2b-1		

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4.6.2.2.3c Skewed Bridges

The following shall replace the second paragraph of A4.6.2.2.3c.

In determining end shear for beams other than prestressed concrete adjacent box beams, the shear skew adjustment factor shall be applied to the shear distribution factor of exterior beams at the obtuse corners for a distance of one-half the span length (see Figure 4.6.2.2.3cP-1).

In determining end shear for prestressed concrete adjacent box beams, the shear skew adjustment factor shall be applied to the shear distribution factors for all the beams which are on a skew (see Figure 4.6.2.2.3cP-1).

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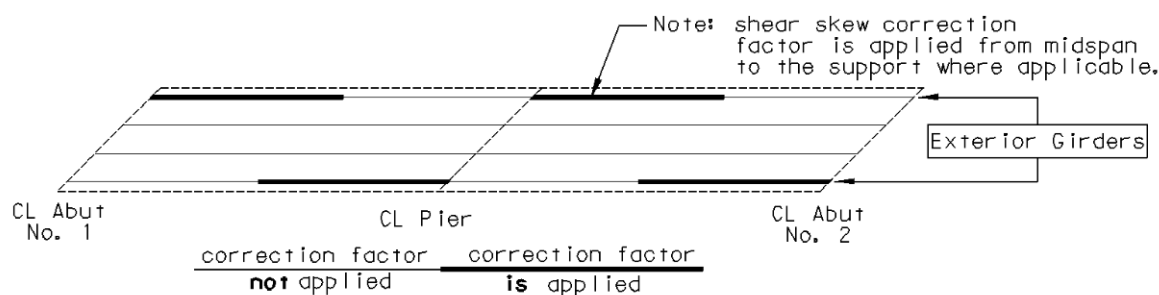
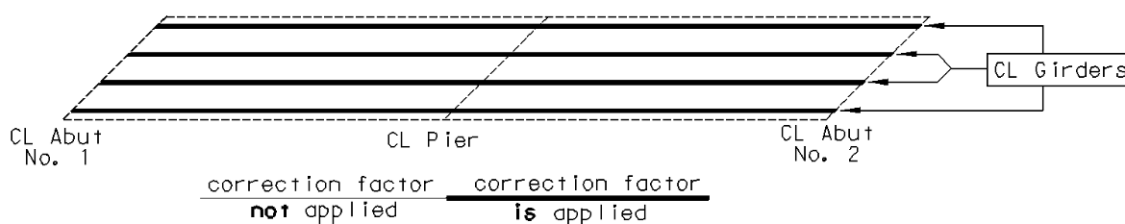
C4.6.2.2.3c.

The following shall supplement AC4.6.2.2.3c.

When structures have multiple skew angles, the smallest applicable skew angle associated with that girder should be used in calculation of shear correction factors.

Steel and P/S I-Beams and P/S Spread Box Beams:

Shear skew correction factor is to be applied to the shear at the ends of exterior beams at the obtuse corner (up to mid-span, where applicable).

**P/S Adjacent Box Beams:**

Note: The examples shown are based on the assumption that the girder is continuous over the pier. If such is not the case (i.e. at an expansion bearing), the correction factor would be applied at the simple end in a similar manner to which it was applied at the abutment.

Figure 4.6.2.2.3cP-1 - Application of Shear Correction Factor for End Shear

In determining end reactions of continuous beams (such as reactions at abutments) for beams other than prestressed concrete adjacent box beams, the shear skew adjustment factor shall be applied to the shear distribution factor of exterior beams at the obtuse corners (see Figure 4.6.2.2.3cP-2).

In determining end reactions of continuous beams (such as reactions at abutments) for prestressed concrete adjacent

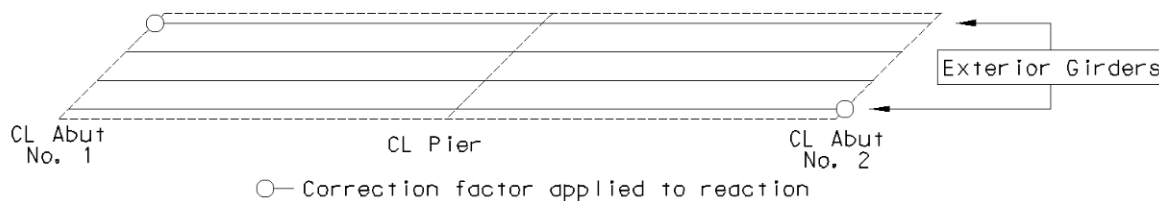
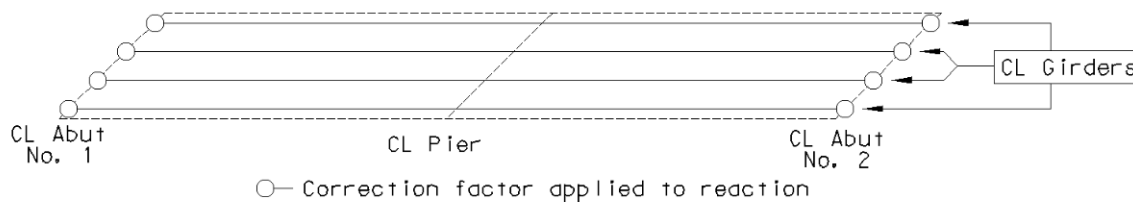
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box beams, the shear skew adjustment factor shall be applied to the shear distribution factors for all the beams which are on a skew (see Figure 4.6.2.2.3cP-2).

Steel and P/S I-Beams and P/S Spread Box Beams:

Shear skew correction factor is to be applied to exterior girder reactions at the obtuse corner of simple-end reactions. It is not applied at continuous reactions such as pier reactions.

**P/S Adjacent Box Beams:**

Note: The examples shown are based on the assumption that the girder is continuous over the pier. If such is not the case (i.e. at an expansion bearing), the correction factor would be applied at the simple end in a similar manner to which it was applied at the abutment.

Figure 4.6.2.2.3cP-2 - Application of Shear Skew Correction Factor for End Reactions

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The following shall replace Table A4.6.2.2.3c-1.

Table 4.6.2.2.3c-1 - Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner

Type of Superstructure	Applicable Cross-Section from Table A4.6.2.2.1-1	Correction Factor	Range of Applicability
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Section or NEXT Beams	a, e, k and also i, j if sufficiently connected to act as a unit	$1.0 + 0.20 \left(\frac{12 L t_s^3}{K_g} \right)^{0.3} \tan(90 - \theta)$	$30^\circ \leq \theta \leq 90^\circ$ $3.5' \leq S \leq 16.0'$ $20' \leq L \leq 500'$ If $L > 240'$, use $L = 240'$ $N_b \geq 4$
Cast-in-Place Concrete Multicell Box	d	$1.0 + \left[0.25 + \frac{12 L}{70 d} \right] \tan(90 - \theta)$	$30^\circ \leq \theta \leq 90^\circ$ $6.0' < S \leq 13.0'$ $20' \leq L \leq 500'$ If $L > 240'$, use $L = 240'$ $35'' \leq d \leq 110''$ $N_b \geq 3$
Concrete Deck on Spread Concrete Box Beams	b, c	$1.0 + \frac{\sqrt{L d}}{6 S} \tan(90 - \theta)$	$30^\circ \leq \theta \leq 90^\circ$ $6.0' \leq S \leq 11.5'$ If $S > 11.5'$, use $S = 11.5'$ $20' \leq L \leq 500'$ If $L > 140'$, use $L = 140'$ $17'' \leq d \leq 66''$ $N_b \geq 3$
Concrete Box Beams Used in Multi-beam Decks	f, g	$1.0 + \frac{12 L}{90 d} \sqrt{\tan(90 - \theta)}$	$30^\circ \leq \theta \leq 90^\circ$ $20' \leq L \leq 500'$ If $L > 120'$, use $L = 120'$ $17'' \leq d \leq 66''$ $35'' \leq b \leq 60''$ $5 \leq N_b \leq 20$

4.6.2.2.4 Curved Steel Bridges

The following shall supplement A4.6.2.2.4

Use of approximate methods must be approved by the Chief Bridge Engineer at the TS&L stage.

4.6.2.5 Effective Length Factor, K

The following shall supplement A4.6.2.5.

For Extreme Event I, Seismic Loading, the effective length factor, K, in the plane of bending may be assumed to be unity in the calculation of λ .

4.6.2.6 Effective Flange Width

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4.6.2.6.1 General

The following shall replace the last sentence of the first paragraph.

For the calculation of live load deflections, where required, the provisions of D2.5.2.6.2 shall apply.

C4.6.2.6.1

The following shall supplement AC4.6.2.6.1.

For typical continuous span bridges, the effective span length may be taken as the distance between points of dead load contraflexure, and can be approximated as shown in Table C4.6.2.6.1P-1 below:

Table C4.6.2.6.1P-1 - Approximate Effective Span Lengths for Computing Effective Slab Width of Continuous Span Bridges

Positive Flexure Regions		Negative Flexure Regions		
Exterior Span	Interior Span	Support Not Adjacent to Exterior Span	For Two-Span Girder	Support Adjacent to Exterior Span
$0.7 L_E$	$0.5 L_I$	$0.25 L_{I1} + 0.25 L_{I2}$	$0.3 L_{E1} + 0.3 L_{E2}$	$0.30 L_E + 0.25 L_I$

Where L_I and L_E represent interior and exterior span lengths, respectively. This approximation is only applicable for the purpose of computing effective span lengths to be used in the determination of the effective slab width. The approximation is based on assumed points of dead load contraflexure for balanced span lengths. In cases where this simplification is not valid, i.e., the spans are not balanced, a more rigorous estimation of the effective span length should be made by determining the actual points of dead load contraflexure. Because the effective slab width affects the section properties to be used in the girder dead load analysis, an iterative procedure may be required to determine the appropriate values. In such cases, a 5% tolerance is allowed on the effective slab width, b_s .

Where the cross-section of the web or top flange changes over a flexural region, the minimum top flange width or web thickness within that region may be used in the calculation of the effective slab width.

Following the determination of the actual points of dead load contraflexure by the continuous beam analysis, the designer should compute the effective slab width in accordance with A4.6.2.6.1 based on the true effective span length. If the effective span length controls the calculation of b_s , the calculated value should be compared to the previously computed value to determine whether it is within the allowed tolerance. The effective span length will typically control the calculation of b_s only for short-spans having large girder spacings.

For steel girders, an acceptable approximation of the permanent load inflection points is the non-composite dead load contraflexure points based on the non-composite section properties.

For prestressed concrete girders, an acceptable approximation of the permanent load inflection points is the

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4.6.2.10 Equivalent Strip Widths for Box Culverts*4.6.2.10.2 Case 1: Traffic Travels Parallel to Span**C4.6.2.10.2*

composite dead load contraflexure points based on the composite (3n) section properties.

The following shall supplement AC4.6.2.10.2
For the BXLRFD computer program, Equation A4.6.2.10.2-2 has been ignored.

*4.6.2.10.4 Precast Box Culverts**C4.6.2.10.4*

The following shall supplement A4.6.2.10.4.

For precast box culverts, the distribution width computed with Equation A4.6.2.10.2-1 shall not exceed the length between two adjacent joints without a means of shear transfer across the joint. Additionally, if no means of shear transfer is provided, the section ends shall be designed as an edge beam in accordance with the provisions of A4.6.2.1.4b.

Shear transfer may be provided by pavement, soil backfill, or a physical connection between adjacent sections.

The following shall supplement AC4.6.2.10.4.

Most shallow cover box culvert applications have some fill or a pavement that likely provide sufficient shear transfer to distribute live load to adjacent box sections without shear keys to avoid higher stresses due to edge loading. States and design agencies that utilize grouted shear keys, pavement or systems whose function is to transfer shear across joints may use past performance of these connections and/or materials as a basis for providing adequate shear transfer. Otherwise, for applications with zero depth of cover, and no pavement, soil, or other means of shear transfer such as shear keys, designers should design the culvert section for the specified reduced distribution widths. The use of post-tensioning in accordance with the Standard Drawing BC-798M in conjunction with a cast-in-place slab or bituminous pavement is considered sufficient to provide adequate shear transfer between adjacent culvert sections.

4.6.2.11P Girder - Floorbeam - Stringer Bridges*4.6.2.11.1P Girder Live Load Distribution Factors*

Girder live load distribution factors shall be calculated based on the assumption that the deck acts as a simple span between the girders or the deck acts as beam with overhang for exterior girders (i.e., this assumes the stringers and floorbeams are not present).

4.6.2.11.2P Stringer Live Load Distribution Factors

Stringer live load distribution factors shall be based on D4.6.2.2 and A4.6.2.2.

*4.6.2.11.3P Floorbeam Live Load Distribution Factors**4.6.2.11.3aP Floorbeams with the Top Flange not Directly Supporting the Deck*

For floorbeams with the top flange not directly supporting the deck, the longitudinal reaction of design live load is determined and then these loads are moved transversely along the floorbeam to produce the maximum

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force effect assuming the stringers are not present.

4.6.2.11.3bP Floorbeams with the Top Flange Directly Supporting the Deck

For floorbeams with the top flange directly supporting the deck, the floorbeam distribution shall be calculated as given in A4.6.2.2.

4.6.2.12P Distribution of Load from the Superstructure to the Substructure

In order to determine girder reactions (which are used as loads for the substructure design), the deck is assumed to act as a simple beam between interior girders and as a cantilever beam for the exterior girder and the first interior girder.

In the calculation of live load girder reactions, the design vehicle shall be assumed to spread uniformly over 10 ft. (i.e., not two separate wheel loads).

4.6.3 Refined Methods of Analysis

4.6.3.1 General

The following shall replace the second paragraph of A4.6.3.1.

Barriers shall not be considered in the calculation of the structural stiffness nor structural resistance of a structure.

The following shall supplement A4.6.3.1.

When a refined method of analysis is performed for beam-slab bridges, other than those bridges defined in D4.6.1.2.1 and D4.6.2.2.1 (which must use a refined method analysis), the capacity of the beam resulting from the refined analysis must not be less than the capacity of the beam designed using the approximate method of analysis given in A4.6.2.2.1 and D4.6.2.2.1.

4.6.3.2 Decks

4.6.3.2.3 Orthotropic Plate Model

The following shall replace A4.6.3.2.3.

In orthotropic plate modeling, the flexural rigidity of the elements may be uniformly distributed along the cross-section of the deck. Where the torsional stiffness of the deck is not contributed solely by a solid plate of uniform thickness, the torsional rigidity should be established by physical testing, three-dimensional analysis, or generally accepted and verified approximations, and shall be approved by the Chief Bridge Engineer.

C4.6.2.12P

For abutments designed on a per foot basis, an acceptable alternate method of distribution would be to divide the sum total of all the loads applied to the abutment (for each limit state) by the abutment front face width. When using the above approach, the live load contribution may be obtained by determining the live load effect for one lane of loading and multiplying that effect by the number of design lanes and the multiple presence factor.

C4.6.3.1

Delete the second paragraph of AC4.6.3.1.

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4.6.3.3 Beam-Slab Bridges**4.6.3.3.1 General**

The following shall supplement A4.6.3.3.1.

When a refined method of analysis is performed, the live load force effects carried by each girder may be computed by the techniques listed below in order of decreasing sophistication:

- Three-dimensional finite element method, and
- Two-dimensional grillage analogy

With a two-dimensional grillage analogy or a three-dimensional finite element method, the rating of a bridge is not as straightforward as when the A4.6.2.2 or D4.6.2.2 is used. A refined method of analysis more closely represents the fact that the distribution of live loads on a bridge is not described by a constant distribution factor. When a refined method of analysis is used in the design, a table of live load distribution factors (based on design truck of the PHL-93) for girder maximum positive and negative moments and shear in each span shall be provided on the contract plans to aid in future ratings of the bridge. The live load distribution factor shall be in the form of a ratio of the force effect from the refined method of analysis caused by the design truck of the PHL-93 in that lane, divided by the force effect obtained from application of one design truck of the PHL-93 acting on a single, isolated girder. The commentary provides an example table of live load distribution factors.

Two other areas which must be considered for future ratings of bridges designed with refined method analysis are cross-frame forces and uplift at reactions. Therefore, at Type, Size and Location submission, the designer must include (for approval by the Chief Bridge Engineer) a proposed simplified method for rating a special vehicle for the controlling cross-frame member and uplift reaction condition.

When a grillage analogy or finite element model is implemented, the preferred procedure is the generation and subsequent loading of an influence surface to produce maximum effect. The influence surface shall be loaded to maximize positive and negative design values (moments, shear, diaphragm forces, etc.) for all critical points along the bridge. This process is analogous to the classical use of influence lines. The provisions of A3.6.1.1.2 shall apply to the loading method described above. It is also acceptable, though less rigorous and economical, to apply distribution factors from D4.6.2.2.2 to each girder in a grid system and allow the grid or finite element analysis to more accurately determine the load distribution to interior and exterior beams, and to account for the influences of skew and curvature, if present. The latter procedure is less preferred than the former procedure since uneconomical designs may result.

It is expressly *not* acceptable to load a grid or finite

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C4.6.3.3.1

The following shall supplement the bulleted list of AC4.6.3.3.1.

- If the program being used allows only for nodal loads, concentrated loads shall be distributed to adjacent nodes by simple statics. If the spacing of girders significantly exceeds 8 ft., it is preferable to place intermediate nodes on the transverse members to model load distribution more accurately.
- The framing of members at bearings is very important. Nodes at bearings should not be artificially restrained through the enforcement of fixed support conditions for other than vertical transitional support at all bearings, and longitudinal and transverse transitional support where the detailing dictates. This provision is critical to proper modeling of bridges with significant skew.
- The warping stiffness of girders is difficult to model, unless a warping (not St. Venant torsional) degree of freedom is explicitly provided by the computer program. As an approximation, the dead load moment diagram and live load moment envelope may be converted to a uniform flange bending (i.e., bimoment) loading by the approximation below:

$$W = \frac{\sum M}{Rd}$$

where:

ΣM = total primary moment kip-in/ft

R = radius (ft.)

d = girder depth (in.)

This load may be applied to the flange and assumed to be reacted by the diaphragms and slab, as appropriate, analogous to a continuous beam on simple supports. If a warping degree of freedom or some approximate method of calculating warping stress is not included in the program, it will be necessary to combine the loads calculated according to the above method with the loads produced from diaphragms.

- If grid or finite element methods are used to determine dead load forces, then the structure, including diaphragms, must be cambered in accordance with the computer analysis. If some cambering other than that corresponding to the opposite of the natural dead load deflection is anticipated (e.g., cambering for more efficient dead load distribution), it must be reflected in the computer modeling.

The following shall supplement AC4.6.3.3.1.

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element model with the number of design lanes of live load given in A3.6.1.1.1, without simultaneously positioning the loads longitudinally and transversely to maximize and minimize each design moment, shear, diaphragm load (curved and/or skewed bridges), and reaction. Except in the most obvious cases (e.g., bridges that are essentially right and straight), the transverse and longitudinal position of live load which produces the critical design condition should be determined by the use of influence surfaces.

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Special care shall be exercised in modeling, analysis and interpretation of the results of two- and three-dimensional procedures. AC4.6.3.3.1 and this commentary provide guidelines. The designer is responsible for the correct application of advanced analysis methods and is advised that various commercial and generic computer programs can report significantly different results for various combinations of skew and/or curvature.

Figure C4.6.3.3.1P-1 provides an example of a table for live load distribution factors which shall be included on the contract drawings. Since the figure provides only example tables, the table used on the contract drawings shall be developed for the specific structure in question.

When a refined method of analysis is used to design a bridge, the table of live load distribution factors shall be the basis for future ratings of the bridge. In order to provide a realistic rating in the absence of such a table, the rating of the bridge would require modeling the bridge by means of an analysis method similar to the one used in the original design. The live load distribution factors given in D4.6.2.2.2 should not be used for the rating, since they would generally provide an unduly conservative rating or, in a few cases, an unconservative rating.

Girder 1-Moment Distribution Factors*						
Location	Positive Moment Distribution Factors			Negative Moment Distribution Factors		
	1 Lane	2 Lanes	N Lanes	1 Lane	2 Lanes	N Lanes
Span 1				–	–	–
Interior Support 1	–	–	–			
Span 2				–	–	–
Interior Support 2	–	–	–			
Span N				–	–	–
Interior Support N	–	–	–			

Girder 1 – Shear Distribution Factors*			
Location	1 Lane	2 Lanes	N Lanes

*These tables shall be repeated for each girder

Figure C4.6.3.3.1P-1 - Example of Live Load Distribution Factor Tables Required for Refined Analyses

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A live load distribution factor developed for the design truck may be used for permit and rating vehicles. When the bridge or the vehicle or both are unusual, an analysis should be made to justify the use of a design truck distribution factor for other types of vehicles.

One possible way to develop a simplified method for rating for cross-frame members would be to develop a table of ratings for PHL-93, P-82, ML-80, HS20 and H20 for 1 up to N lanes loaded at the time of original design. When a future rating of a special vehicle is required, the engineer could develop an approximate rating by interpolating among these vehicles used to develop the table.

A similar approach could be used for the uplift reaction condition. A table of uplift reactions for PHL-93, P-82, ML-80, HS20 and H20 for 1 up to N lanes loaded could be developed at the time of original design. When a future rating of a special vehicle is required, the engineer could develop an approximate uplift reaction by interpolating among these vehicles used to develop the table. Next, this uplift reaction could be used in computing a rating.

Some programs use an influence surface loading technique that evaluates numbers of lanes of loading with appropriate multi-presence reduction factors. Other procedures apply the distribution factors given in D4.6.2.2.2 to each girder in a grid of girders and diaphragms and allow a grid analysis to evaluate the effects of skew, curvature, and the tendency for load to gravitate toward the exterior stringers, but they do not necessarily take advantage of the fact that the true distribution factor is often less than those given in D4.6.2.2.2 and typically is not constant for all effects (moment, shear, deflection, etc.) or locations along the bridge.

In 1986, the Department conducted a parametric study of steel girder bridges to compare the results obtained from these programs for a variety of combinations of skew and curvature. This parametric study could not cover all cases that might be encountered by designers, but did reveal some trends and comparative data which form the basis of a report titled "Review of Computer Programs for the Analysis of Girder Bridges", January 1989. Since 1986, many of the programs used in this study have been changed and/or updated. Therefore, some of the report's comparisons and observations may have lost their relevancy. For a copy of this report, contact the Chief Bridge Engineer's office.

The modeling of diaphragms and boundary conditions at supports and bearings is vital to obtaining the proper results from some of these sophisticated programs. The burden of correctly handling these factors rests squarely upon the shoulders of the designer. Consider the following example, which shows how a very small modeling error produces very erroneous results.

The framing plan shown in Figure C4.6.3.3.1P-2 represents a real bridge that was designed using a grid-type approach. The designer had a good model for this structure, except that the rotational degree of freedom corresponding to the global x-axis at all of the bearings was fixed. This did

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not allow the diaphragms at the piers and abutments to respond correctly to the imposed loadings and deformations, and also had the effect, by virtue of vector resolution between global and local systems, of producing artificially stiff ends on the girders.

This incorrect boundary assumption altered both the dead load and live load forces. The effect of this on the dead load reactions obtained at the abutments and piers was dramatic. A modest uplift was reported at the near abutment dead load reaction for Girder 1, and a very substantial uplift was reported at the far abutment dead load reaction for Girder 5. This is shown in the table in Figure C4.6.3.3.1P-2, as is a moment diagram for noncomposite dead loads which reflects the incorrect dead load reactions. Also shown are the correct dead load reactions which were determined when the structure was modeled using the generic STRESS computer program, with proper boundary conditions at the supports. In this case, a positive dead load reaction was found at all bearings, and a significantly different moment diagram for noncomposite dead load also resulted. Finally, the reaction table shows a set of incorrect dead load reactions obtained with STRESS when same error in boundary conditions was made to confirm the significance of proper modeling. The correct and incorrect dead load moment diagrams also are shown in Figure C4.6.3.3.1P-2.

The differences in the degrees of freedom at the lines of support on this structure were also investigated, utilizing a relatively complete three-dimensional finite element analysis and the SAPIV computer program. The model is illustrated in Figure C4.6.3.3.1P-3, which shows how the deck slab, girders and cross-frames were modeled in their proper relative positions in cross-section by means of rigid linking members. Also shown in this figure is a comparison of the dead load reactions obtained from STRESS and from SAPIV by applying all the noncomposite dead loads in a single loading. The agreement between these reactions is excellent.

In order to verify that the order of pouring the deck slab units would not contribute to an uplift situation, the pouring sequence was replicated in a three-dimensional SAPIV analysis. The results of the analysis of the three stages of the pouring sequence are shown in Figure C4.6.3.3.1P-3, as well as the total accumulated load at the end of the pour. A comparison of the sequential loading with the application of a single loading of noncomposite dead load also showed relatively good agreement.

There are cases in which dead load uplift at the reactions due to skew and/or curvature is possible. The simple span bridge featured in the November 1, 1984, issue of Engineering News-Record was correctly analyzed and indicated high corner dead load uplift.

The important point shown in the example in Figures C4.6.3.3.1P-2 and C4.6.3.3.1P-3 is that seemingly small errors in modeling the structure can result in very substantial changes in the reactions, shears and moments. The designer must be aware of this potential when using

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two- or three-dimensional analysis techniques.

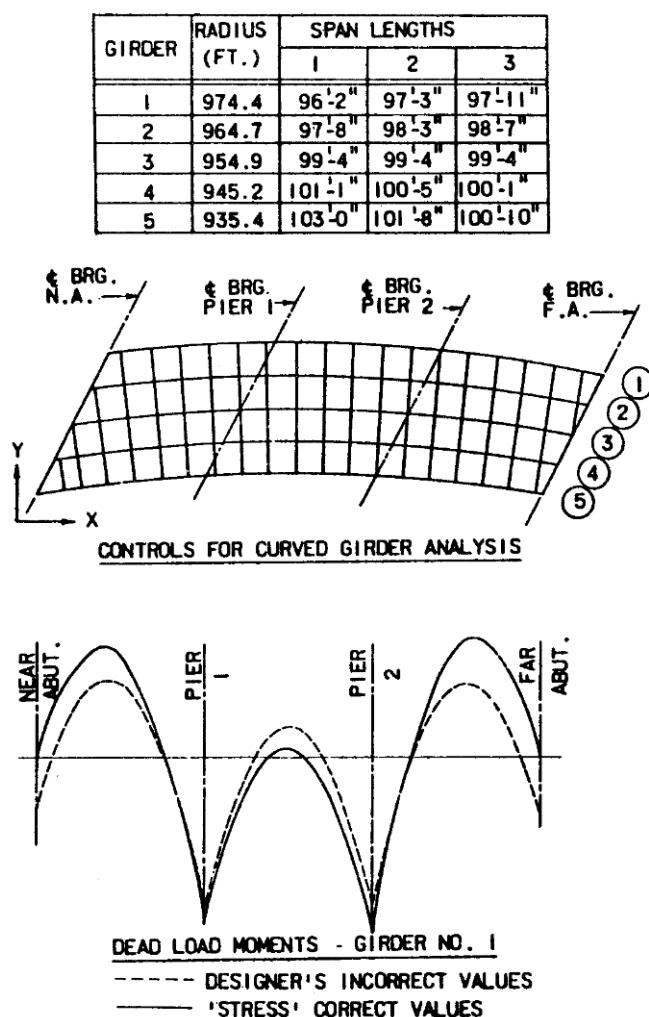
Sometimes modeling problems occur because user's manuals are not clear or because a "bug" exists, of which the author/vendor is unaware. Such a case is illustrated for a simply supported, partially curved and skewed bridge in Figure C4.6.3.3.1P-4. Initially, this bridge was modeled with extra joints at locations other than diaphragms in an effort to improve live load determination. As a result, the number of points along each girder was not equal, but there was no indication of a potential problem in the descriptive literature. The resulting live load moment envelopes for the middle and two exterior girders, which are shown in Figure C4.6.3.3.1P-4a, are obviously unusual in shape and possibly also in their order of maximum moment, i.e., No. 4, No. 5 and No. 3.

After these results were brought to the attention of the authors/vendors, it was decided that the live load processor was not responding properly to the unequal number of nodes per girder, that nodes should be essentially "radial," and that it was uncertain whether the nodes to which diaphragms were not connected were legitimate. The revised model, shown in Figure C4.6.3.3.1P-4b, produced clearly better results, as is shown in the indicated live load moment envelopes. This example illustrates the need to review the output carefully and to communicate with the authors/vendors about the correct use of their programs.

In summary, two- and three-dimensional methods can provide better understanding of load distribution in girder bridges, but must be used with great care.

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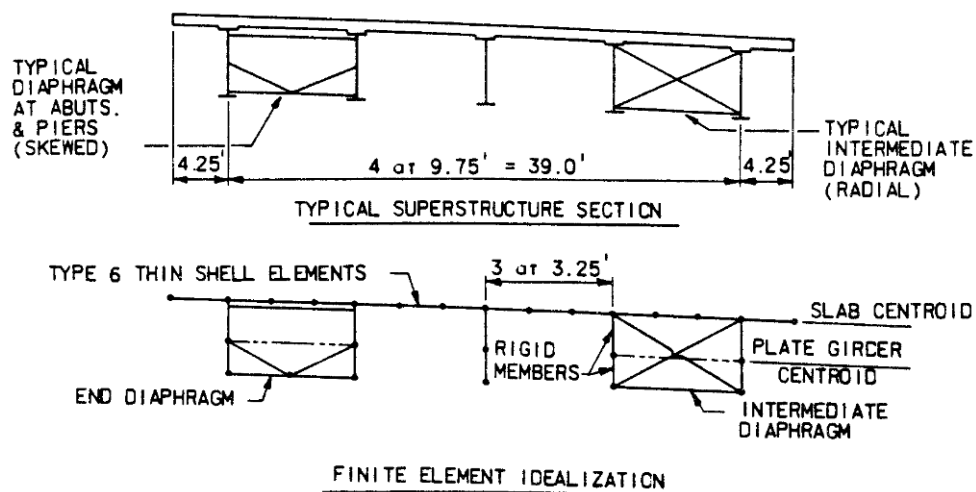


DEAD LOAD SUPPORT REACTIONS				
GIRD. NO.	'STRESS' CORRECT SUPPORT CONDITIONS	DESIGNER'S INCORRECT REACTIONS	'STRESS' INCORRECT SUPPORT CONDITIONS	
	(VERT - K)	(VERT - K)	(VERT - K)	(MOM X K-FT)
NEAR ABUT.	1	64.93	-18.07	-17.78
	2	67.47	90.32	90.22
	3	65.10	88.75	88.72
	4	64.88	58.84	58.76
	5	75.33	168.83	168.33
PIER 1	1	207.73	190.90	191.24
	2	193.43	184.99	184.21
	3	186.21	178.74	178.93
	4	208.42	198.36	198.47
	5	168.15	164.77	164.19
PIER 2	1	188.74	180.23	180.33
	2	202.24	191.43	190.56
	3	199.25	184.03	184.34
	4	201.38	185.16	185.25
	5	184.37	171.21	170.65
FAR ABUT.	1	66.68	223.62	223.25
	2	64.36	47.07	47.14
	3	64.96	86.29	86.49
	4	66.62	117.95	117.95
	5	69.46	-81.63	-81.51
TOTAL	2609.71	2611.79	2609.73	

Figure C4.6.3.3.1P-2 - Framing Plan, Comparative Dead Load Reactions and Moment Diagram Showing Effect of Proper and Improper Rotational Boundary Condition as Reflected in a Grid Analysis

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	BEAM NO	SAP NCODE	SAP FULL DEAD LOAD ON NON-COMPOSITE SECTION	STRESS FULL DEAD LOAD ON NON-COMPOSITE SECTION	SAP SLAB POURS 1 & 2	SAP SLAB POUR 3	SAP SLAB POUR 4	SAP TOTAL OF COLUMNS 3, 4 & 5
NEAR ABUT.	5	8	53.7	56.5	57.7	-4.2	-0.4	53.1
	4	29	61.1	58.4	65.6	-1.5	0.2	64.3
	3	64	56.0	55.5	59.8	-4.5	-0.5	54.8
	2	109	57.4	55.1	61.9	-2.3	-0.3	59.3
	1	167	65.2	65.4	73.4	-13.0	-2.9	57.5
PIER 1	5	282	180.2	185.4	94.4	22.5	60.4	177.3
	4	324	164.1	169.1	67.1	33.3	62.2	162.6
	3	363	171.7	163.1	74.7	33.0	67.0	174.7
	2	402	174.0	183.4	88.2	28.2	61.2	177.6
	1	444	149.9	151.4	64.1	29.4	60.9	154.4
PIER 2	5	576	171.3	166.9	74.3	36.6	66.4	177.3
	4	617	172.2	178.1	84.1	26.5	60.8	171.4
	3	656	176.7	174.0	84.0	30.4	64.9	179.3
	2	695	169.5	176.7	74.9	28.3	61.9	165.1
	1	736	165.7	165.1	73.4	25.7	63.6	162.7
FAR ABUT.	5	901	57.6	58.2	63.6	-11.3	-2.4	49.9
	4	935	55.3	54.7	58.7	-1.9	0.2	57.0
	3	959	55.8	55.4	59.7	-4.1	0.1	55.7
	2	979	58.4	57.5	64.2	-3.6	0.1	60.7
	1	991	58.9	60.3	64.7	-4.0	-0.6	60.1
			2275	2290				2275

Figure C4.6.3.3.1P-3 - Finite Element Idealization and Reactions Obtained for Structure Shown in Figure C4.6.3.3.1P-2

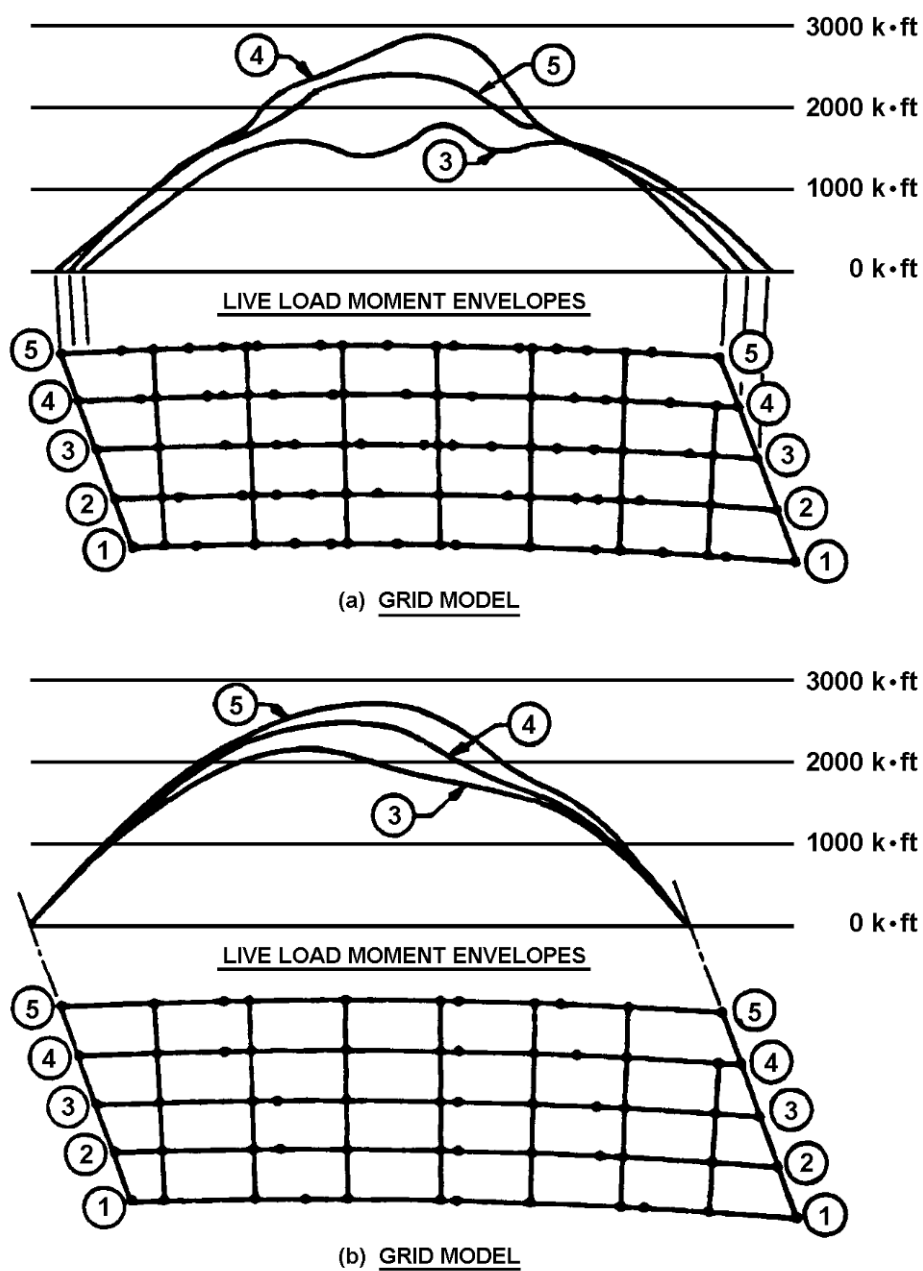


Figure C4.6.3.3.1P-4 - Comparative Live Load Moment Envelopes for the Middle and Two Outside Girders of Curved Skewed Showing the Results of an Apparent "Bug" in Algorithm for Applying Live Load

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4.6.3.3.2 Curved Steel Bridges

The following shall supplement A4.6.3.3.2

Refined analysis methods shall be used for the analysis of curved steel bridges unless an approximate method has been approved by the Chief Bridge Engineer at the TS&L stage.

4.6.4 Redistribution of Negative Moments in Continuous Beam Bridges**4.6.4.1 General**

The following shall replace A4.6.4.1.

The redistribution of force effects in multi-span, multi-beam or girder superstructures is not permitted in the design of Pennsylvania bridges.

4.6.4.2 Refined Method

Delete A4.6.4.2

4.6.4.3 Approximate Procedure

Delete A4.6.4.3

4.7 DYNAMIC ANALYSIS**4.7.1 Basic Requirements of Structural Dynamics****4.7.1.4 Damping****C4.7.1.4**

The following shall supplement AC4.7.1.4.

Damping values obtained from field measurements or tests shall be approved by the Chief Bridge Engineer.

4.7.2 Elastic Dynamic Responses**4.7.2.2 Wind-Induced Vibration***4.7.2.2.1 Wind Velocities*

The following shall supplement A4.7.2.2.1.

The Chief Bridge Engineer will decide if wind tunnel tests are warranted for a structure.

*4.7.2.2.3 Design Considerations**C4.7.2.2.3*

The following shall supplement AC4.7.2.2.3.

Wind-induced vibration of two vertical members on a three-span continuous deck truss was observed during a routine inspection. Specifically, 'no load' verticals were found to be vibrating excessively in the wind conditions at the time.

It was determined that one of the verticals was vibrating at a frequency very close to that calculated for the first weak

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axis flexural mode determined by finite element analysis (FEA). As the frequency of vortex shedding approaches one of the natural frequencies of the structure, significant amplitudes of vibrations can be caused by a condition termed “lock-in”. The frequency of vortex shedding, f_s , is given by Equation C4.7.2.2.3P-1.

$$f_s = 1.47 \frac{SU}{D} \quad (\text{C4.7.2.2.3P-1})$$

where:

S = Strouhal number

U = wind velocity (mph)

D = cross section dimension in direction perpendicular to wind (ft.)

The frequency of vortex shedding is directly related to the Strouhal number, which is a function of the cross-section shape. For I-shaped members, this value can vary greatly depending on the method used to obtain it. Therefore, special consideration must be given in determining the appropriate Strouhal number (i.e. perform a sensitivity study to compare results of various Strouhal numbers).

The study performed to analyze the above mentioned vertical concluded that the simplified method using closed-form equations was in agreement with that of the frequency measured in the field as well as the frequency determined by the FEA. The flexural response of the truss vertical was found to be most closely represented by a fixed-fixed beam with the modal frequencies, f_i , given by Eq C4.7.2.2.3P-2.

$$f_i = \frac{\lambda_i^2}{2\pi L^2} \sqrt{\frac{EIg}{w}} \quad (\text{C4.7.2.2.3P-2})$$

where:

λ_i = parameter for fixed-fixed mode “i” ($\lambda_1 = 4.73$)

E = modulus of elasticity (ksi)

I = moment of inertia (in.⁴)

g = acceleration of gravity (in./sec²)

L = free length of truss vertical between gusset plate edges plus two times the flange width (in.)

w = weight per unit length (kip/in.)

Using the first modal frequency, the predicted critical wind velocity for the vertical was determined to be approximately 20 mph. A steady wind velocity of this magnitude is plausible at the bridge; therefore, vortex shedding is considered the likely cause of the wind-induced vibration phenomena observed at the bridge.

4.7.4 Analysis for Earthquake Loads

4.7.4.3 Multi-Span Bridges

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4.7.4.3.1 *Selection of Method*

The following shall supplement A4.7.4.3.1.

Based on the General Procedure, Pennsylvania is classified as Seismic Zone 1. Detailed seismic analysis is not required for single span bridges or bridges in Seismic Zone 1, unless the site soils are classified as either Site Class E or F.

A seismic analysis may be performed for multi-span bridges in Seismic Zone 1, for non Site Class E or F soils, if designers feel such an analysis will more accurately reflect the connection forces and produce a more economical design.

4.7.4.4 **Minimum Support Length Requirements**

The following shall replace the definition of N in Equation A4.7.4.4-1.

N = minimum support length measured perpendicular to abutment or pier face from the end of the beam at the centerline of the bottom flange.

The following shall supplement A4.7.4.4.

The N calculated in Equation A4.7.4.4-1 shall not be taken less than 12 in..

NOTE: S in Equation A4.7.4.4-1 is based on AASHTO's definition of skew angle, see PP3.2.2 for PennDOT's and AASHTO's definition of skew angle.

The following shall replace Table A4.7.4.4-1.

ZONE	SITE CLASS	%N
1	A,B C,or D	100
1	E	125
1	F	150

4.7.4.6P **Base Isolation Design**4.7.4.6.1P *General*

The use of Base Isolation design must be approved by the Chief Bridge Engineer at the TS&L stage.

Refer to the AASHTO Guide Specifications for Seismic Isolation Design, 3rd Edition, July 2010 for base isolation design guidance.

COMMENTARY

C4.7.4.3.1

The following shall supplement AC4.7.4.3.1.

WinSEISAB should be used for the seismic analysis. If WinSEISAB is not suitable for a structure, other structural analysis programs capable of seismic modeling (such as STAAD) may be used in lieu of WinSEISAB. The Designer must stipulate the reasons WinSEISAB is not suitable and obtain the Department's approval.

C4.7.4.6.1P

The benefits of seismic isolation for bridges may be summarized as follows:

- Reduction in the realistic forces to which a bridge will be subjected by a factor of between 5 and 10.
- Elimination of the ductility demand and, hence, damage to the piers.
- Control of the distribution of the seismic forces to the substructure elements with appropriate sizing of the elastomeric bearings.

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- (d) Reduction in column design forces by a factor of approximately 2 in comparison with conventional design.
- (e) Reduction in foundation design forces by a factor of between 2 and 3 in comparison with conventional design.

The intent of seismic isolation design is to eliminate or significantly reduce damage (inelastic deformation) to the substructure.

4.8 ANALYSIS BY PHYSICAL MODELS**4.8.2 Bridge Testing**

The following shall replace A4.8.2.

When approved by the Chief Bridge Engineer, existing bridges may be instrumented in accordance with Publication 238, Bridge Safety Inspection Manual, Part IE, Chapter 5.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

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5.2 DEFINITIONS

The following shall supplement A5.2.

Lightweight Concrete - Concrete containing lightweight aggregate and having an air dry unit weight not exceeding 115 pcf.

Special Anchorage Device - Anchorage device whose adequacy should be proven in a standardized acceptance test and approved by Chief Bridge Engineer prior to its usage. Most multi-plane anchorages and all bond anchorages are Special Anchorage Devices.

5.3 NOTATION

The following shall replace the notations of f'_c and f'_{ci} in A5.3.

f'_c = compressive structural design strength of concrete at 28 days, unless another age is specified (ksi) (D5.4.2.1)

f'_{ci} = compressive structural design strength of concrete at time of initial loading or transfer (ksi) (D5.9.4.4P)

5.4 MATERIAL PROPERTIES

5.4.1 General

The following shall replace the first and second paragraphs of A5.4.1.

Designs should be based on the material properties cited herein and on the use of materials which conform to the standards for the grades of construction materials as specified in Publication 408.

When other grades or types of materials are used, their properties, including statistical variability, shall be established prior to design and approved by the Chief Bridge Engineer. The minimum acceptable properties and test procedures for such materials shall be specified in the contract documents.

5.4.2 Normal Weight and Structural Lightweight Concrete

5.4.2.1 Compressive Strength

The following shall replace A5.4.2.1.

Minimum mix design compressive strength (ksi) shall be in accordance with Section 704.1(b) of Publication 408.

The following classes of cement concrete with corresponding f'_c and n values are to be used for structural designs:

C5.4.2.1

The following shall replace AC5.4.2.1.

The strength requirements in Publication 408 for mix design strength are intended to ensure strengths within a 95% confidence level.

For structures designed before August 16, 1989, and had Class AAA concrete specified in the design, use 4.5 ksi as the structural design strength for analysis. For structures designed after August 16, 1989, and had Class AAA concrete specified in the design, use 4.0 ksi as the structural design strength for analysis. For the design of all new structures, use AAAP concrete with a structural design strength of 4.0 ksi.

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Class of Cement Concrete	f'_c Structural Design Strength	Normal Weight n Modular Ratio	Lightweight n Modular Ratio
AAAP	4.0 ksi	8	12
AAA	4.0 ksi	8	12
AA	3.5 ksi	8	13
A	3.0 ksi	9	14
C	2.0 ksi	11	17

The weights for normal weight and lightweight concrete are given in A3.5.1 and D3.5.1, respectively.

The use of different classes of cement concrete shall be as follows:

Class AAAP

- deck slab
- precast channel beams
- sidewalks
- top slab of concrete box culverts at grade

Class AAA

- precast channel beams

Class AA

- curbs
- barriers
- divisors
- concrete diaphragms
- abutment backwalls
- cheek walls
- shear blocks
- U-wings above bridge seat construction joint
- footings if needed to resist high diagonal tension stresses
- sound barriers

Class A

- piers
- abutments below bridge seat
- pedestals
- wingwalls
- retaining walls
- footings
- arch culverts
- spandrel walls
- walls and top and bottom slabs of box culverts under fill
- walls and bottom slab of box culverts at grade
- caissons

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Class C

- below bottom of footings when specified

Cement concrete for precast prestressed concrete bridge members and other precast components shall comply with the requirements of Publication 408.

Prestressed concrete members shall be designed with a structural design strength (f'_c) between 5 ksi and 10 ksi. For use of structural design strength (f'_c) greater than 8 ksi, there must be a clear economic advantage to be gained. Justification for utilizing structural design strength greater than 8 ksi must be submitted at the TS&L stage for approval. For design-build projects and alternate designs for concrete structures submitted by the Contractors, the use of concrete strengths greater than 8 ksi for the prestressed concrete members will be permitted without further justification.

Precast deck panels (when permitted) and precast box culverts shall be designed with a structural design strength (f'_c) of 5 ksi.

All prestressed components not otherwise specified shall have a structural design strength of 5 ksi unless approval is obtained from the Chief Bridge Engineer.

Lightweight concrete shall not be used for prestressed applications, or concrete box culverts.

The use of lightweight concrete may be considered for deck slabs on rehabilitation projects where reduction of weight is important.

Show the structural design strength (f'_c) of the concrete for each part of the structure on the plans.

5.4.2.2 Coefficient of Thermal Expansion

The following shall replace A5.4.2.2.

The thermal coefficient of expansion shall be taken as:

- for normal weight concrete: $6.0 \times 10^{-6}/^{\circ}\text{F}$, and
- for lightweight concrete: $5.0 \times 10^{-6}/^{\circ}\text{F}$

If a more precise coefficient of thermal expansion is required, laboratory tests shall be performed on the specific mix to be used. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

5.4.2.3 Shrinkage and Creep**5.4.2.3.1 General**

The following shall supplement A5.4.2.3.1.

For the design of single span prestressed beams, creep

It is preferred to use only lightweight coarse aggregate in the mix for which the unit weight of the concrete will be 0.115 kcf. It is possible to obtain a concrete with a unit weight of only 0.105 kcf by using lightweight aggregate for both the coarse and fine aggregate, but it is difficult to control such a mix during construction.

C5.4.2.3.1

The following shall supplement AC5.4.2.3.1.

The Department's design procedure for prestressed concrete beams neglects the axial effects induced by creep and shrinkage.

In the past, the Department has neglected creep and

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and shrinkage effects shall be neglected.

For the design of prestressed beams, creep and shrinkage effects shall only be applied when they cause adverse effects.

5.4.2.3.2 *Creep*

The following shall replace the definitions of t and H in the first paragraph of A5.4.2.3.2.

t = maturity of concrete, i.e., time since casting (day)

H = relative humidity (%), shall be taken as 70%

The following shall replace the last paragraph of A5.4.2.3.2.

The surface area used in determining the volume to surface area ratio should include only the area that is exposed to atmospheric drying. For prestressed concrete beams, the top surface (beam/slab interface) shall be included in surface area used in determining the volume to surface area ratio (see DC5.4.2.3.2). For prestressed concrete box beams, the surface area of the void shall not be included in determining the volume to surface ratio (see DC5.4.2.3.2).

5.4.2.3.3 *Shrinkage*

The following shall replace the definition of t in A5.4.2.3.2 for shrinkage.

t = drying time after moist or steam curing (day)

5.4.2.4 **Modulus of Elasticity**

The following shall supplement A5.4.2.4.

f'_c Structural Design Strength (ksi)	Normal Weight E_c Modulus of Elasticity (ksi)	Lightweight E_c Modulus of Elasticity (ksi)
4.0	3,600	2,600
3.5	3,400	2,400
3.0	3,100	2,200
2.0	2,600	1,800

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shrinkage effects for single span prestressed beams without any adverse effects. Therefore, the Department will continue this practice.

C5.4.2.3.2

The following shall supplement AC5.4.2.3.2.

The volume to surface area ratio to be used in the determination of creep and shrinkage effects shall take into consideration the condition of the member during the time period where the creep and shrinkage effects are most significant. For example, the engineer may wish to consider the entire perimeter of a prestressed beam being exposed if the majority of the shrinkage will have taken place prior to the placement of the slab. However, the surface area of the slab itself would not include the portion of the slab of the beam/slab interface.

For Equations AC5.4.2.3.2-1 and AC5.4.2.3.2-2, if the volume to surface (v/s) ratio is greater than 6 in., use v/s ratio of 6 in..

The Department's experience with prestressed concrete box beams has shown that the void in box beam is not adequately ventilated. Therefore, the surface area of void is not considered in the calculation of volume-to-surface area ratio.

C5.4.2.4

The following shall supplement AC5.4.2.4.

The equation for calculation of E_c is based on the unit weight of concrete without an allowance for steel reinforcement. Therefore, for normal weight concrete (0.150 kcf) and lightweight concrete (0.115 kcf), w_c equals 0.145 kcf and 0.110 kcf, respectively, in Equation A5.4.2.4-1.

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5.4.2.5 Poisson's Ratio

The following shall replace A5.4.2.5.

Poisson's ratio shall be assumed to be 0.2. If a more precise Poisson's ratio is required, laboratory tests shall be performed on the specific mix to be used. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

For components which are expected to be subject to cracking, the effect of Poisson's ratio may be neglected.

5.4.2.6 Modulus of Rupture

The following shall supplement A5.4.2.6.

If a more precise modulus of rupture is required, laboratory tests shall be performed on the specific mix to be used. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

5.4.3 Reinforcing Steel**5.4.3.1 General**

The following shall replace A5.4.3.1.

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric and welded deformed wire fabric shall conform to the materials standards as specified in Publication 408.

Reinforcement bars shall be designed with $f_y = 60$ ksi. ASTM-A 615/A 615M, ASTM-A 996/A 996 M, and ASTM-A 706/A 706M, Grade 60, reinforcement steel shall be used. Reinforcement bars with $f_y = 40$ ksi may only be used with approval of the Chief Bridge Engineer. When approved by the Chief Bridge Engineer, Grade 40 bars shall comply with ASTM-A 615/A 615M, Grade 40.

Welding of reinforcement bars during fabrication or construction will not be allowed unless specified or permitted by the Chief Bridge Engineer. When specified or permitted by the Chief Bridge Engineer, welding of the Grade 60 bars shall be preceded by preheat according to applicable construction specifications. If specified or approved by the Chief Bridge Engineer, welded splices or other mechanical connections may be used according to the AASHTO Specifications.

Deformed reinforcement shall be used, except plain bars or plain wire may be used for spirals, hoops and wire fabric.

Rail steel (ASTM-A 996/A 996 M), reinforcement bars shall not be used in the design of bridge piers, abutments, beams, footings and piles where ductility is essential for structural performance during seismic activities. Furthermore, this steel shall not be used where bending or welding of the reinforcement is required. The rail steel (ASTM-A 996/A 996 M) reinforcement bars may be used for deck and diaphragms. The above restrictions shall be clearly indicated on both the design drawings and shop

C5.4.3.1

The following shall supplement AC5.4.3.1.

The ductility in reinforcement bars used in portions of superstructures and substructures is a design consideration pertaining to seismic loadings. ASTM-A 996/A 996 M is a tough, but brittle material and does not provide sufficient ductility to satisfy seismic design requirements.

The barrier design equations are based on the yield line theory, and ductile failure of reinforcement steel is required.

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drawings by adding the last sentence of Note 6 in PP1.7.4 to the general note sheet and the individual bar schedule.

5.4.3.3 Special Applications

The following shall supplement the second paragraph of A5.4.3.3.

The use of ASTM A1035 reinforcement is only permitted with Chief Bridge Engineer approval.

The following shall supplement A5.4.3.3.

Requirements for which substructure and superstructure components are required to have epoxy-coated reinforcement are given in D5.4.3.6P.

5.4.3.4P Contract Documents**C5.4.3.4P**

Detailing of bars shall be such that the total length shall not exceed 40 ft. for No. 3 and No. 4 bars, and 60 ft. for larger bars and 8 ft. in projections from the primary direction.

Bar marks should be simple and instructive as to number, size and sequence of placement, e.g., 21-F502@18". In general, the following key letters are recommended:

- In stem of abutment, wings, and retaining walls W
- In piers P
- In deck slab S
- In diaphragms D
- In stems of T-beams B
- In curbs and barriers C
- In approach slabs AS
- In sleeper slabs SS
- In moment slabs MS
- In footings (including dowels) F

Use the prefix E to designate epoxy-coated bars, e.g., 21-ES502@6".

The number of bars and spacing shall be shown only at one place--in plan, section, or elevation, whichever is best. At other locations, show only size and mark.

The bar schedule shall be arranged in tabular form for each part of the structure (Pier 1, Pier 2, deck slab, etc.), indicating the bar mark, size, length and type (straight or bent) of each bar. The bending diagram shall be shown near the bar schedule. Preferably in the contract drawings, bar schedules for substructure and superstructure components shall be segregated to assist in field usage (e.g., abutment bar schedules should follow abutment drawings, pier bar schedules should follow pier drawings, deck bar schedules should follow deck drawings, etc.).

All lap lengths shall be indicated on the drawings.

Embedment length of all dowels shall be shown on the drawings.

Splices of No. 14 and No. 18 bars shall be mechanical

The 40 ft. and 8 ft. lengths are limits for the purpose of shipping and handling.

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splices and shall be shown on the drawings.

5.4.3.5P Constructibility

To facilitate placement and vibration of concrete in reinforced concrete members over 4 ft. in depth (pier caps, deep footings, etc.), reinforcement shall be designed and detailed to provide the following clearances:

Stirrup spacing--minimum of 9 in. clear, except in a localized region where a smaller spacing will not interfere with placement or vibration.

Top longitudinal bar spacing--9 in. clear in at least one space and 6 in. minimum at all other spaces.

In cases where anchor bolts must be embedded in heavily reinforced concrete members, the reinforcement shall be designed and detailed to allow the contractor the option of placing the anchor bolts in preformed holes or of drilling when permitted. The bars shall be arranged to clear a circle concentric with, and 2 in. larger than, the anchor bolt.

When specifying a dowel hole diameter, use a minimum diameter that is at least two times (2x) the diameter of the dowel bar, i.e. = doweled hole for No. 5 bar has a 1 1/4 in. diameter. Set dowels in predrilled holes filled with non-shrink grout. Do not use dowels for concrete members of thickness less than 10 in..

Bars may be bundled to obtain the required clearance. In no case shall a concrete member be oversized to obtain the required clearance, unless there is no other acceptable alternative.

Except for vertical reinforcement extending from the top of footing into pier columns and wall stems (abutments, wingwalls and retaining walls), the projection of vertical reinforcement bars into an adjacent pour shall not exceed 15 ft.. In such cases, dowels shall be used.

5.4.3.6P Epoxy-Coated Reinforcing Bars

To prevent or minimize the deterioration of structural concrete caused by deicing chemicals, fusion-bonded epoxy-coated rebars shall be provided as follows:

(a) Superstructures

1. All bars in reinforced concrete deck slabs, curbs, barriers and backwalls, including bars that protrude into these elements from some other portion of the structure, shall be epoxy-coated. Diaphragm bars adjoining the expansion dam and protruding into the deck slab shall also be epoxy-coated.
2. As a preventive measure to guard against rusting of steel in prestressed concrete beam ends at joints and subsequent spalling of concrete for all new beams, all mild reinforcing

C5.4.3.6P

The potential for salt spray damage will be greatest at grade separation structures where sprays are generated by traffic below the bridge. In some cases, damaging sprays may be created by wind currents at high-level structures crossing waterways, ravines and other similar natural features.

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steel for a distance of 9 ft. from the beam end shall be fully epoxy-coated, regardless of beam type. This requirement applies only to the beam end adjoining a deck joint regardless of the type of joint dam/seal installed.

(b) Substructures

1. Epoxy-coated rebar and a breathable sealant (as per PP3.4.2) shall be provided in all portions of the substructure above a plane approximately 3 ft. below the finished grade for the following cases:
 - Piers located under expansion joints or exposed to salt spray.
 - Abutments, wingwalls and retaining walls exposed to salt spray.
 - Abutment portions exposed to discharge from troughs located below expansion dams.
 - Abutment stems located below expansion joints.
2. Epoxy-coat main reinforcement bars (J-bars) protruding from abutment, wing and pier footings into the stem or columns.

Epoxy-Coated J-Bars:

In the “Investigation of Cause and Mitigation of J-bar Deterioration in Bridge Structures in Pennsylvania”, July 2011, Modjeski and Masters reports instances of significant section losses in the J-bars at the juncture of the abutment stem and the footer. In most cases, if not all, this area will accumulate moisture since a crack is always present due to the construction joint. The increased usage of drainable backfill increases the probability of continued moisture in this area. Since this bar is the most critical bar in performance of abutments, it is prudent to require the J-bar to be epoxy-coated for all projects.

5.4.4 Prestressing Steel**5.4.4.1 General**

The following shall replace the first portion of the first sentence of A5.4.4.1.

Uncoated low-relaxation seven-wire strand, or uncoated plain or deformed, high-strength bars, shall conform to the following material standards,

The following shall supplement A5.4.4.1.

Low relaxation prestressing steel shall be used, unless an alternative is approved by the Chief Bridge Engineer.

For the use of 0.6 in. diameter strands, there must be a clear economic advantage to be gained. Justification for

C5.4.4.1

Delete AC5.4.4.1.

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utilizing 0.6 in. diameter strands must be submitted at the TS&L stage for approval. For design-build projects and alternate designs for concrete structures submitted by the Contractors, the use of 0.6 in. diameter strands will be permitted without further justification.

The diameter for strands used in prestressed plank beams shall be 3/8 in. or 1/2 in. special. The fabricator may use 1/2 in. special strands, provided the final force per strand does not exceed the as-designed final force for the 3/8 in. strand, i.e., strand-for-strand substitution is permitted.

5.4.5 Post-Tensioning Anchorages and Couplers

The following shall replace the first paragraph of A5.4.5.

Anchorages and tendon couplers shall conform to the requirements of Publication 408.

5.4.6 Ducts**5.4.6.1 General**

The following shall replace A5.4.6.1.

Ducts for tendons shall be rigid or semi-rigid galvanized ferrous metal.

The radius of curvature of tendon ducts shall not be less than 20 ft., except in the anchorage areas where 12 ft. may be permitted.

The effects of grouting pressure on the ducts and the surrounding concrete shall be investigated.

The maximum support interval for the ducts during construction shall be indicated in the contract documents.

5.5 LIMIT STATES**5.5.3 Fatigue Limit State****5.5.3.1 General**

Delete the third paragraph of A5.5.3.1.

COMMENTARY

C5.4.5

Delete the first paragraph of AC5.4.5.

The following shall replace the second paragraph of AC5.4.5.

Characteristics of anchorages and couplers are summarized below.

The following shall replace the third and last bulleted items of AC5.4.5, respectively.

Couplers are to be used only at locations shown on the contract documents or approved by the Chief Bridge Engineer.

Unless waived by the Chief Bridge Engineer because of suitable previous tests and/or experience, qualification of anchorages and couplers are to be verified by testing. The test results must be submitted and approved by the Chief Bridge Engineer prior to usage.

C5.4.6.1

Delete AC5.4.6.1.

C5.5.3.1

Delete the fourth paragraph of AC5.5.3.1.

The following shall supplement AC5.5.3.1.

A5.5.3.1 lists the concrete components that do not have to be checked for the fatigue limit state. Therefore, if the

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specification does not expressly state that the fatigue limit state does not need to be checked for a certain concrete component, it must be checked for that concrete component. The fatigue limit state check for concrete components applies to both superstructure and substructure units.

5.5.3.2 Reinforcing Bars

The following shall supplement A5.5.3.2.

The effective fatigue stress range shall not exceed the constant-amplitude fatigue threshold $(\Delta F)_{TH}$, as defined in equation A5.5.3.2-1 or A5.5.3.2-2.

$$\begin{aligned} f_{fe} &= \text{effective fatigue stress range in reinforcing steel (ksi)} \\ &= \gamma \text{ (PTF)} \Delta f_s \end{aligned}$$

where:

PTF = Pennsylvania Traffic Factor as given in Table D6.6.1.2.2-1

γ = load factor specified in Table A3.4.1-1 for the Fatigue I load combination

Δf_s = calculated unfactored fatigue stress range in reinforcing steel (ksi)

5.5.3.3 Prestressing Tendons

The following shall replace A5.5.3.3.

The effective fatigue stress range in prestressing tendons shall not exceed 10 ksi.

$$\begin{aligned} f_{fp} &= \text{effective fatigue stress range in prestressing tendons (ksi)} \\ &= \gamma \text{ (PTF)} \Delta f_p \end{aligned}$$

where:

PTF = Pennsylvania Traffic Factor as given in Table D6.6.1.2.2-1

γ = load factor specified in Table A3.4.1-1 for the Fatigue I load combination

Δf_p = calculated fatigue stress range in prestressing tendon (ksi)

5.5.4 Strength Limit State**5.5.4.2 Resistance Factors****C5.5.3.3**

Supplement AC5.5.3.3 with the following.

Since Pennsylvania uses allowable tensile stress in a precompressed tensile zone equal to $0.0948\sqrt{f'_c}$, it is believed that fatigue is not of concern.

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5.5.4.2.1 *Conventional Construction*

Add the following bullets to the first paragraph:

- for tension-controlled prestressed voided concrete box beams..... 0.95
- for axial resistance of the concrete portion of concrete filled undamaged steel pipe piles in compression:..... $\phi = 0.55$
- for axial resistance of prestressed concrete piles in compression..... $\phi = 0.45$
- for axial resistance of precast concrete piles in compression..... $\phi = 0.45$
- for combined axial and flexural resistance of undamaged concrete filled steel pipe piles
axial resistance $\phi = 0.60$
flexural resistance..... $\phi = 0.85$

Add the following sentence to the beginning of the fourth paragraph of A5.5.4.2.1.

Partially prestressed components shall not be used unless written approval is obtained from the Chief Bridge Engineer.

5.5.4.2.2 *Segmental Construction*

The following shall supplement A5.5.4.2.2.

Unbonded post-tensioning systems shall not be used, except when permitted by the Chief Bridge Engineer for temporary condition, future rehabilitation schemes, or rehabilitation of existing bridges.

The last sentence of the second paragraph of A5.5.4.2.2 shall be changed from "...approved by the Engineer" to "...approved by the Chief Bridge Engineer".

5.6 DESIGN CONSIDERATIONS

5.6.1 General

The following shall supplement A5.6.1.

The value of the moment of inertia for the computation of flexural stiffness of slabs, beams, columns, etc. shall be based on gross concrete section with the effect of reinforcement neglected for computation of forces only.

In the computation of flexural stiffness and resistance moments of beams, the haunch shall be taken as zero. However, in the computation of dead load, the haunch shall be taken into account.

For box culverts, include the effect of the haunches for computation of flexural stiffness.

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C5.5.4.2.1

The following shall supplement AC5.5.4.2.1.

This reduction is to account for the possibility of the void forms floating out of place.

The basis for the resistance factors for concrete and concrete filled piles is described in DC5.13.4.1.

C5.6.1

The following shall supplement AC5.6.1.

The beam haunch depth can vary at the time of construction. Therefore, at the time of design, the Department does not want to count on something that may not be there. However, if the haunch depth of an existing beam can be determined from field measurements, the haunch may be used in the computation of flexural stiffness and resistance moments for the analysis of an existing beam.

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5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS**5.7.1 Assumptions for Service and Fatigue Limit States****C5.7.1**

PSLRFD rounds the modular ratio for reinforced and prestressed concrete to the nearest integer, except that the ratio due to differences between concrete strengths (such as deck to beam) is rounded to the nearest tenth.

5.7.2 Assumptions for Strength and Extreme Event Limit States**5.7.2.1 General**

The following shall replace the fourth and seventh bulleted items in A5.7.2.1, respectively.

- If a maximum strain exceeding 0.003 is to be utilized, the concrete must be confined, test results must show that it is obtainable and it must be approved by the Chief Bridge Engineer.
- The concrete compressive stress-strain distribution is assumed to be rectangular as given in A5.7.2.2. Other assumed concrete compressive stress-strain distribution shapes may be used if they are substantiated with test results and approved by the Chief Bridge Engineer.

5.7.2.2 Rectangular Stress Distribution

The following shall supplement A5.7.2.2.

β_1 shall be based on slab concrete for composite construction.

5.7.3 Flexural Members**5.7.3.1 Stress in Prestressing Steel at Nominal Flexural Resistance****5.7.3.1.1 Components with Bonded Tendons****C5.7.3.1.1**

The following shall replace A5.7.3.1.1.

For rectangular or flanged sections subjected to flexure about one axis where the approximate stress distribution specified in A5.7.2.2 is used and for which f_{pe} is not less than $0.5 f_{pu}$, the average stress in prestressing steel, f_{ps} , may be taken as:

$$f_{ps} = f_{pu} \left(1 - k \frac{c}{d_p} \right) \quad (5.7.3.1.1-1P)$$

for which:

The following shall supplement AC5.7.3.1.1.

The equation for c has been developed so that it can accommodate the neutral axis in the slab, the flange or the web. The LRFD equations for c only accommodate the neutral axis in the flange or in the web.

For ease of computation, the equation for c was developed with structural design strength of the slab as the basis. Therefore, the width of beam flange and the width of web have to be transformed using the ratio of structural design strengths. Since this is an ultimate condition, the concrete is in the inelastic range and the typical modular ratio based on the modulus of elasticity for the slab and beam concrete does not apply.

Figure D5.7.3.2.6P-1 provides a graphic representation

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$$k = 2 \left(1.04 - \frac{f_{py}}{f_{pu}} \right) \quad (5.7.3.1.1-2P)$$

of some of the notations.

$$c = \frac{A_{ps} f_{pu} - 0.85 \beta_1 f'_c [b' h_f + b_s t_s - b_w' (h_f + t_s)]}{0.85 \beta_1 f'_c b_w' + k A_{ps} \frac{f_{pu}}{d_p}} \quad (5.7.3.1.1-3P)$$

$$b' = b \left(\frac{f'_{c \text{ beam}}}{f'_c} \right) \quad (5.7.3.1.1-4P)$$

$$b_w' = b_w \left(\frac{f'_{c \text{ beam}}}{f'_c} \right) \quad (5.7.3.1.1-5P)$$

where:

- A_{ps} = area of prestressing steel (in.²)
- f_{pu} = specified tensile strength of prestressing steel (ksi)
- f_{py} = yield strength of prestressing steel (ksi)
- A_s = area of mild steel tension reinforcement (in.²)
- f'_c = structural design strength of the slab concrete (ksi)
- $f'_{c \text{ beam}}$ = structural design strength of the beam concrete (ksi)
- f_y = yield strength of tension reinforcement (ksi)
- b = width of beam top flange (in.)
- b' = transformed width of beam top flange (in.)
- b_s = effective flange width (in.)
- b_w = width of web (in.)
- b'_w = transformed width of web (in.)
- h_f = depth of beam top flange (in.)
- t_s = depth of deck (in.)
- d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.)
- c = distance between the neutral axis and the compressive face (in.)

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β_1 = stress block factor specified in A5.7.2.2

For a composite beam with the neutral axis in the deck,
 $b' = b_s$ and $b'_w = b_s$.

For a composite beam with the neutral axis in the beam
top flange, $b'_w = b'$.

For a composite beam with the neutral axis in the web,
no modifications.

For a non-composite beam, f'_c = structural design
strength of the beam concrete.

For a non-composite beam with the neutral axis in the
beam top flange, $t_s = 0$, $b_s = 0$ and $b'_w = b'$.

For a non-composite beam with the neutral axis in the
web,
 $t_s = 0$ and $b_s = 0$.

5.7.3.1.2 Components with Unbonded Tendons

The following shall supplement A5.7.3.1.2.

This article applies to tendons which are unbonded
full-length and not partial length debonded strands used in
prestressed beams. Full-length unbonded tendons are not
allowed unless approved by the Chief Bridge Engineer.

5.7.3.2 Flexural Resistance

5.7.3.2.2 Flanged Sections

The following shall supplement A5.7.3.2.2.

The nominal flexural resistance of prestressed beams
shall be determined using D5.7.3.2.6P.

5.7.3.2.5 Strain Compatibility Approach

The following shall supplement A5.7.3.2.5

Strain compatibility approach shall not be used unless
approved in writing by the Chief Bridge Engineer.

5.7.3.2.6P Prestressed Beam Flexural Resistance

C5.7.3.2.6P

For prestressed beams subjected to flexure about one
axis, where the appropriate stress distribution specified in
A5.7.2.2 is used and the tendons are bonded, the general
equation for nominal flexural resistance shall be taken as:

$$M_n = T \left(d_p - \frac{t_s}{2} \right) - C_1 \frac{h_f}{2} - C_2 \left(\frac{\beta_1 c}{2} - \frac{t_s}{2} \right) \quad (5.7.3.2.6P-1)$$

for which:

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$$C_0 = 0.85 f'_c (b_s - b') t_s \quad (5.7.3.2.6P-2)$$

$$C_1 = 0.85 f'_c (b' - b'_w) (t_s + h_f) \quad (5.7.3.2.6P-3)$$

$$C_2 = 0.85 f'_c b'_w \beta_1 c \quad (5.7.3.2.6P-4)$$

$$T = A_{ps} f_{ps} \quad (5.7.3.2.6P-5)$$

The definitions for the notations in the above equations are given in the list in D5.7.3.1.1.

Equation 5.7.3.2.6P-1 applies to Figure 5.7.3.2.6P-1. For a composite beam with the neutral axis in the deck, $b' = b_s$ and $b'_w = b_s$.

For a composite beam with the neutral axis in the beam top flange, $b'_w = b'$.

For a composite beam with the neutral axis in the web, no modifications.

For a non-composite beam, f'_c = structural design strength of the beam concrete.

For a non-composite beam with the neutral axis in the beam top flange, $t_s = 0$, $b_s = 0$ and $b'_w = b'$.

For a non-composite beam with the neutral axis in the web, $t_s = 0$ and $b_s = 0$.

COMMENTARY

The location of C_0 is where the moment was summed about in the development of Equation 5.7.3.2.6P-1.

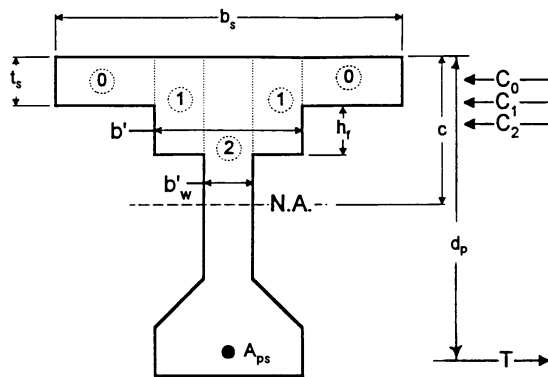


Figure 5.7.3.2.6P-1 - General Beam Cross-Section

5.7.3.3 Limits for Reinforcement

5.7.3.3.1 Maximum Reinforcement

The following shall supplement A5.7.3.3.1.

Over-reinforced prestressed components, as described in AC5.7.3.3.1 ($c/d_e > 0.42$), shall not be used unless approved by the Chief Bridge Engineer.

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5.7.3.3.2 Minimum Reinforcement

The following shall replace A5.7.3.3.2.

The minimum reinforcement of any section of a flexural component, reinforced with conventional rebars, prestressing tendons or any combination thereof, is that adequate to develop a factored flexural resistance, M_r , at least equal to the lesser of:

- 1.2 times the cracking moment, M_{cr} , determined on the basis of elastic stress distribution and the modulus of rupture, f_r , of the concrete as specified in D5.4.2.6, or
- 1.33 times the factored moment required by the applicable strength load combinations specified in A3.4.1 and D3.4.1.

For prestressed concrete girders, the resistance provided by the prestressing reinforcement must satisfy the minimum reinforcement provisions for positive flexure at all points along the beam, excluding the ends of the beam in the development zone.

The provisions for shrinkage and temperature reinforcement of A5.10.8 and D5.10.8 must also be considered.

The cracking moments for prestressed beams in the positive moment region shall be as given below:

Non-composite

$$M_{crnc} = M_D + S_{bc} \left[\frac{P}{A_g} + \frac{Pe}{S_b} - \frac{M_{DNF}}{S_b} - \frac{M_D}{S_{bc}} + f_r \right] \quad (5.7.3.3.2-2P)$$

Composite

$$M_{crc} = M_{DNF} + M_{DCF} + S_{bc} \left[\frac{P}{A_g} + \frac{Pe}{S_b} - \frac{M_{DNF}}{S_b} - \frac{M_{DCF}}{S_{bc}} + f_r \right] \quad (5.7.3.3.2-3P)$$

Composite-Transformed

$$M_{crct} = M_{DNF} + M_{DCF} + S_{bct} \left[\frac{P}{A_g} + \frac{Pe}{S_b} - \frac{M_{DNF}}{S_b} - \frac{M_{DCF}}{S_{bc}} + f_r \right] \quad (5.7.3.3.2-4P)$$

where:

- P = final prestressing force (kips)
- e = eccentricity of p/s force, measured from the neutral axis to the centroid of the p/s force (in.)

COMMENTARY

C5.7.3.3.2P

In the design of prestressed beams, the creep and shrinkage effects do not need to be taken into account for the cracking moment.

For prestressed beams made continuous, this provision should also be satisfied for both the positive and negative moment connections.

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COMMENTARY

f_r	=	modulus of rupture of the concrete (ksi)
M_D	=	dead load, final moment (kip-in.)
M_{DNF}	=	dead load, non-composite, final moment (kip-in.)
M_{DCF}	=	dead load, composite, final moment (kip-in.)
S_b	=	bottom sections modulus, non-composite, non-transformed (in. ³)
S_{bc}	=	bottom section modulus, composite, transformed for slab (in. ³)
S_{bct}	=	bottom section modulus, composite, transformed for slab and prestressing steel (in. ³)

In the calculation of S_{bc} , the slab is transformed by modular ratio of E_{slab}/E_{beam} .

In the calculation of S_{bct} , the slab and the prestressing steel in the beam are transformed by the modular ratio of E_{slab}/E_{beam} and $E_{p/s}/E_{beam}$, respectively.

5.7.3.4 Control of Cracking by Distribution of Reinforcement

The following shall replace the definition of d_c in A5.7.3.4.

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.). The (1/2 in.) wearing surface for deck slab, top and bottom slab of box culvert and (1 in.) extra cover provided to account for uneven ground level of footing bottom mat reinforcement and bottom slab of box culverts, shall not be included.

The following shall supplement the third paragraph of A5.7.3.4.

Class 1 applies to all reinforced concrete members except precast and cast-in-place box culverts, segmental construction and for the specific conditions defined under Class 2. Class 2 exposure also applies to precast and cast-in-place box culverts.

The following shall replace the fourth paragraph of A5.7.3.4.

In the computation of d_c , the actual concrete cover thickness is to be used except in deck slabs, box culvert slabs and footings as defined in d_c .

5.7.3.6 Deformations

5.7.3.6.2 Deflection and Camber

C5.7.3.6.2

The following shall supplement AC5.7.3.6.2.

Equation A5.7.3.6.2-1 applies to members where it is appropriate to consider them as cracked at the service load levels. Therefore, Equation A5.7.3.6.2-1 would apply to reinforced concrete members, but not prestressed concrete members.

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5.7.3.6.4P *Camber of Prestressed Beams*5.7.3.6.4aP *Camber Due to Prestressing*

Camber due to prestressing shall be calculated by Equations 5.7.3.6.4aP-1, 5.7.3.6.4aP-2 and 5.7.3.6.4aP-3 for beams with straight, draped and debonded strands, respectively.

Straight Strands

$$\Delta_{\text{prestressed}} = \frac{P e_s L^2}{8 E_{ci} I} \quad (5.7.3.6.4aP-1)$$

Draped Strands

$$\Delta_{\text{prestressed}} = \frac{P L^2 [4 X^2 (e_n - e_s) + 3 e_s]}{24 E_{ci} I} \quad (5.7.3.6.4aP-2)$$

Debonded Strands

$$\Delta_{\text{prestressed}} = \frac{1}{8 E_{ci} I} [P_b e_b L^2 + P_1 e_1 [L^2 - (L_t + 2L_1)^2] + P_2 e_2 [L^2 - (L_t + 2L_2)^2] + \dots P_i e_i [L^2 - (L_t + 2L_i)^2]] \quad (5.7.3.6.4aP-3)$$

for which:

$$P = P_t \left(1 - \frac{\Delta f_s}{100} \right) \quad (5.7.3.6.4aP-4)$$

$$P_{b;1,2,\dots,i} = P_{t\,b;1,2,\dots,i} \left(1 - \frac{\Delta f_s}{100} \right) \quad (5.7.3.6.4aP-5)$$

where:

e_s = eccentricity at mid-span (in.)

e_n = eccentricity at end of beam (in.)

e_b = eccentricity at mid-span of full-length bonded strands (in.)

$e_{1,2,\dots,i}$ = eccentricity at mid-span of debonded group 1, 2...i (in.)

E_{ci} = modulus of elasticity of beam concrete at transfer (ksi)

I = moment of inertia of beam (in.⁴)

L = beam length (in.)

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COMMENTARY

L_t = transfer length (in.)

$L_{1,2,...i}$ = distance from centerline of bearing to debonding cutoff points

P = prestressing force at selected time for camber calculations (kips)

P_b = prestressing force at selected time for camber calculations of full-length bonded strands (kips)

P_t = prestressing force at transfer (kips)

P_{tb} = prestressing force at transfer of full-length bonded strands (kips)

$P_{b1,2,...i}$ = prestressing force at selected time for camber calculations of debonded group 1, 2...i

$P_{tb1,2,...i}$ = prestressing force at transfer of debonded group 1, 2...i

Δf_s = assumed percentage of prestressing loss since transfer for selected time

X = percent of L for drape point

5.7.3.6.4bP Deflection Due to Dead Loads

The maximum downward deflection at mid-span due to the beam weight and internal diaphragms shall be taken as:

$$\Delta_{D1} = \frac{5M_{D1}L^2}{48E_{ci}I} \quad (5.7.3.6.4bP-1)$$

where:

M_{D1} = unfactored moment at mid-span due to the beam weight and any internal diaphragms (kip-in.)

The maximum downward deflection at mid-span due to slab, formwork, external diaphragms and any other dead load which is applied to the beam before the slab has hardened shall be taken as:

$$\Delta_{D2} = \frac{5(M_{D2})L^2}{48E_cI} \quad (5.7.3.6.4bP-2)$$

where:

M_{D2} = unfactored moment at mid-span due to dead load applied to the beam before the slab has hardened, except the beam weight and internal diaphragms (kip-in)

E_c = modulus of elasticity of beam concrete (ksi)

For simple span construction, the maximum downward deflection at mid-span due to superimposed dead load shall be taken as:

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$$\Delta_{D3} = \frac{5(M_{D3})L^2}{48 E_c I_c} \quad (5.7.3.6.4bP-3)$$

where:

M_{D3} = unfactored moment at mid-span due to superimposed dead load (kip-in.)

I_c = moment of inertia of composite beam (in.⁴)

For continuous span construction, the maximum downward deflection at mid-span due to superimposed dead load shall be determined from continuous span analysis.

5.7.3.6.4cP Total Camber at Transfer of Prestressing

The total camber at transfer shall be taken as:

$$\Delta_t = \Delta_{\text{prestressed}} - \Delta_{D1}$$

Δf_s shall be assumed to be zero in determining $\Delta_{\text{prestressed}}$.

5.7.3.6.4dP Camber for Bearing Slope

The camber for determining the bearing slope shall be taken as:

$$\Delta_b = \Delta_{\text{prestressed}} - \Delta_{D1}$$

Δf_s shall be assumed to be 10% in determining $\Delta_{\text{prestressed}}$.

5.7.3.6.4eP Total Camber in Beams at Time of Construction

The total camber in the beams at time of construction shall be taken as:

$$\Delta_c = (\Delta_{\text{prestressed}} - \Delta_{D1}) C_r$$

where:

$$C_r = 1.6$$

$$\Delta f_s = 10\% \text{ in determining } \Delta_{\text{prestressed}}$$

5.7.3.6.4fP Final Camber

Negative final camber (sag) shall be limited to $L/2000$.

C5.7.3.6.4eP

After release, beams may be stored for a period of several days to possibly six months or more. During this time, the camber increases due to creep. The prestressing force, on the other hand, decreases due to shrinkage, creep of the concrete and relaxation of the steel. These are opposing effects.

Assuming the beams are stored from 7 to 80 days, it may be reasonable to estimate that the creep factor, C_r , varies in a range of 1.5 to 2.0 and the prestress loss, Δf_s , varies in a range of 5 to 15% in that time. For design, unless better information is available, $C_r = 1.6$ and $\Delta f_s = 10\%$ will be used. These are average values from Pennsylvania prestressers. The assumed values used for C_r and Δf_s shall be shown on the design drawings.

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5.7.4 Compression Members**5.7.4.2 Limits for Reinforcement**

The following shall replace the last paragraph of A5.7.4.2.

For bridges in Seismic Zone 1, the minimum area of longitudinal reinforcement may be that required for a component with a reduced effective area of concrete, provided that both the full section and the reduced effective section are capable of resisting the factored loads and that the area of reinforcement is not less than 0.7 percent of the gross area of the column or 1.0 percent of the reduced effective area of the column, whichever is less.

5.7.4.3 Approximate Evaluation of Slenderness Effects

The following shall supplement A5.7.4.3.

To account for reduced levels of cracking in the reinforced concrete members under service load conditions, use an effective moment of inertia that is 1.43 times the moment of inertia given by Equation A5.7.4.3-1 or Equation A5.7.4.3-2 for all serviceability computations.

5.7.4.4 Factored Axial Resistance

The following shall supplement A5.7.4.4.

For computing factored axial resistance, spiral reinforcement not meeting the requirements of Equation A5.7.4.6-1 shall be considered tie reinforcement.

5.7.4.6 Spirals and Ties**C5.7.4.2**

Delete the last two paragraphs of AC5.7.4.2.

C5.7.4.3

The following shall supplement AC5.7.4.3.

The 1.43 multiplier was taken from Section 10.11 of the ACI 318-05 Code.

C5.7.4.6P

The actual ratio, r_s , of spiral reinforcement to total volume of concrete may be computed using simple geometry:

$$r_s = [A_s (2\pi R)] / \text{Volume of concrete within spiral} = 2A_s / (RS)$$

where:

- A_s = Spiral bar area (in.²)
- R = the radius of the spiral steel (in.)
- S = pitch of the spiral (in.)

The volume of concrete is computed as that within the confines of a single spiral loop = $\pi R^2 S$. (in.³)

5.8 SHEAR AND TORSION**5.8.2 General Requirements****5.8.2.9 Shear Stress on Concrete****C5.8.2.9**

The following shall supplement AC5.8.2.9.

The PAPIER and VBent programs will use the lower limit of 0.72h for d_v since the PAPIER and VBent programs

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5.8.3 Sectional Design Model**5.8.3.2 Sections Near Supports**

The following shall replace the second paragraph of A5.8.3.2.

Where the reaction force in the direction of the applied shear introduces compression into the end region of a member, the location of the critical section for shear shall be taken as the larger of $0.5 d_v \cot \theta$ or d_v from the internal face of the support, where d_v and θ are measured at the critical section for shear.

5.8.3.4 Procedures for Determining Shear Resistance**5.8.3.4.1 Simplified Procedure for Nonprestressed Sections****C5.8.3.4.1**

The following shall supplement AC5.8.3.4.1.

Currently, the computer programs ABLRFD, PAPIER and VBent are based on using the simplified procedure.

5.8.3.4.2 General Procedure**C5.8.3.4.2**

The following shall replace the first paragraph of A5.8.3.4.2.

The determination of parameters β and θ shall follow the provisions in AASHTO Appendix B5.

The following shall supplement AC5.8.3.4.2.

Replacing $(V_u - V_p) \cot \theta$ with $(V_u - V_p)$ is meant to eliminate the need for iteration. This simplified approach is acceptable. However, the PSLRFD computer program is based on the iterative approach included in earlier versions of the AASHTO-LRFD Specifications.

In Pennsylvania, the prestressing steel is not continuous over the supports in the typical method used for making prestressed beams continuous for live load.

For this typical method, the prestressing steel near the supports is on the compression side of the beam. Therefore, for AASHTO Appendix B5 Equations B5.2-2 and B5.2-3, A_{ps} will be zero and A_s will be the longitudinal reinforcement in the slab for locations near a continuous support.

5.8.3.5 Longitudinal Reinforcement**C5.8.3.5**

The following shall replace the definition of ϕ_f , ϕ_v and ϕ_c in A5.8.3.5.

ϕ_f , ϕ_v , ϕ_c , ϕ = resistance factors taken from Articles A5.5.4.2 and D5.5.4.2 as appropriate for moment, shear and axial resistance

The following shall supplement AC5.8.3.5.

Equation A5.8.3.5-1 was developed to check the tensile capacity of the reinforcement due to the combined effects of moment, shear and axial force. Therefore, when using Equation A5.8.3.5-1, the following items should be considered: the absolute value of V_u may be used, M_u is taken as positive if it causes tension in the longitudinal reinforcement, M_u is taken as 0.0 if it causes compression in the tensile reinforcement and N_u is taken as positive if it causes tension in the longitudinal reinforcement, N_u is taken as 0.0 if it causes compression in the longitudinal

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5.8.3.6 Sections Subjected to Combined Shear and Torsion**5.8.3.6.3 Longitudinal Reinforcement**

The following shall replace Equation A5.8.3.6.3-1.

$$(A_s f_y + A_{ps} f_{ps}) \geq \frac{M_u}{\phi d_v} + \frac{0.5 N_u}{\phi} + \cot \theta \sqrt{(V_{hcb})^2 + \left(\frac{0.45 p_h T_u}{2 A_o \phi} \right)^2} \quad (5.8.3.6.3-1)$$

where:

$$\text{If } \frac{V_u}{\phi} > 0.5 V_s + V_p$$

$$V_{hcb} = \frac{V_u}{\phi} - 0.5 V_s - V_p$$

$$\text{If } \frac{V_u}{\phi} \leq 0.5 V_s + V_p$$

$$V_{hcb} = 0.0$$

5.8.4 Interface Shear Transfer-Shear Friction**5.8.4.1 General**

The following shall replace the second paragraph of A5.8.4.1.

Reinforcement for interface shear between concretes of slab and beams or girders may consist of single bars, multiple leg stirrups or the vertical legs of welded wire fabric.

5.8.4.2 Computation of the Factored Interface Shear Force, V_{ui} , for Girder/Slab Bridges

The following shall replace the fourth paragraph of A5.8.4.2.

For beams and girders, the longitudinal spacing of the rows of reinforcing bars for interface shear shall not be greater than 21 in..

5.8.4.3 Cohesion and Friction Factors

The following shall modify bulleted items in A5.8.4.3.

The value for cohesion, c , shall be taken as $c = 0.100$ ksi for the first, third and fourth bulleted categories.

reinforcement.

Equation A5.8.3.5-2 is similar to equation A5.8.3.5-1 except that at simply supported ends the value of both the applied moment and applied axial force is 0.0.

C5.8.3.6.3

The following shall supplement AC5.8.3.6.3.

The signs of M_u , V_u and N_u are defined in DC5.8.3.5.

C5.8.4.1

The following shall supplement AC5.8.4.1.

The upper limits in AASHTO equations A5.8.4.1-4 and A5.8.4.1-5, provide capacities based on a 1/4" amplitude surface roughness and are larger than the controlling equation A5.8.4.1-3 (approximately three times greater).

C5.8.4.3

The following shall supplement AC5.8.4.3.

The Department reduced the cohesion values since the minimum amplitude specified in Publication 408, Section

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The value for cohesion, c , shall be taken as $c = 0.150$ ksi for the second bulleted category.

The value for the friction factor, μ , shall be taken as $\mu = 0.85$ for the third bulleted category.

5.8.4.4 Minimum Area of Interface Shear Reinforcement

The following shall replace A5.8.4.4.

The cross-sectional area, A_{vf} , of the reinforcement shall not be less than either that required by Equation A5.8.4.1-3 or $0.019 \text{ in}^2 \text{ per in.}$

5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.1 General Design Considerations

5.9.1.1 General

The following shall supplement A5.9.1.1.

Drape points are usually at a distance $1/3$ of span length measured from the centerline of the bearings. The minimum distance between drape points is 20 ft.. In box beams, internal diaphragms shall be located at the drape points.

5.9.1.4 Section Properties

The following shall replace A5.9.1.4.

For section properties prior to bonding of post-tensioning tendons, effects of loss of area due to open ducts shall be considered.

For both pretensioned or post-tensioned members after bonding of tendons, section properties shall be based on:

- gross section for permanent loads
- transformed section for live loads

The transformed section for live loads shall include the prestressing steel, but the mild steel reinforcement is to be neglected.

5.9.3 Stress Limitations for Prestressing Tendons

The following shall replace Table A5.9.3-1.

COMMENTARY

1107.03(d)5.e.2 is not equivalent to the 0.25" amplitude specified in A5.8.4.3.

C5.8.4.4

The following shall replace AC5.8.4.4.

The Department has historically used a minimum interface shear reinforcement based on a minimum reinforcement area per unit length with a maximum longitudinal reinforcement spacing. The requirements of D5.8.4.2 and D5.8.4.4 are based on this successful past practice.

C5.9.1.1

The following shall supplement AC5.9.1.1.

The Department has a computer program which offers many options for the analysis or design of prestressed concrete beams.

Common sense should be used in matching beam sizes with prestressing forces, while it may be possible, for instance, to stress a 17 in. deep box beam with a 1,000 kips force by increasing f'_c to 8 ksi, the end zone stresses, including secondary stresses, will most likely be excessive and cause distress, evidenced by end zone cracking.

C5.9.3

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Table 5.9.3-1 - Stress Limits for Prestressing Tendons

Condition	Tendon Type	
	Plain High-Strength Bars	Low Relaxation Strand
At Jacking: (f_{pj})		
- Pretensioning	$0.70 f_{pu}$	$0.75 f_{pu}$
- Post-tensioning	$0.76 f_{pu}$	$0.80 f_{pu}$
After Transfer: (f_{pt})		
- Pretensioning	$f_{pj} - \Delta f_{pES}$	$f_{pj} - \Delta f_{pES}$
- Post-tensioning		
• At anchorages and couplers immediately after anchor set	$0.70 f_{pu}$	$0.70 f_{pu}$
• Post-tensioning - General	$0.70 f_{pu}$	$0.70 f_{pu}$
At Service Limit State: (f_{pe})		
After Losses	$\leq 0.80 f_{pj}$	$\leq 0.80 f_{pj}$

The following shall supplement A5.9.3.

For the Service IIIA load combination given in Table D3.4.1.1P-2, the stress in the prestressing steel in the row nearest the extreme tension fiber of the member shall not exceed 90% of the yield stress of prestressing strands.

The upper limit for maximum prestressing force for an as-designed structure is 4,000 kips and resulting end moment must be less than 120,000 kip-in. For alternate designs, the prestressing force can be greater than this, but the safe capacity of the Fabricator's prestressing bed must not be exceeded.

5.9.4 Stress Limits for Concrete

Delete A5.9.4.1, A5.9.4.1.1, A5.9.4.1.2, A5.9.4.2, A5.9.4.2.1, A5.9.4.2.2 and A5.9.4.3.

5.9.4.4P Allowable Concrete Stresses for Pretensioned Beams

A summary of allowable stresses under design loads for pretensioned beams is given in Table 5.9.4.4P-1.

For the service load combinations, which involves traffic loading, tension stresses in members with bonded prestressing tendons should be investigated using Load Combination Service III in Table A3.4.1-1.

The tension in the precompressed tensile zone shall not exceed $0.0948\sqrt{f'_c}$ under any condition.

The flexural stresses at the end of the beams are to be checked at the centerline of bearing and 12 in. from the centerline of bearing towards mid-span. For flexural stresses, the prestressing strands shall be assumed to develop linearly between the end of the beam and the point

The following shall supplement AC5.9.3.

This 90% of the yield stress provision was maintained from Section 5.4.6 of the AASHTO Maintenance Inspection of Bridges Manual (1983, 1990 Interim). Typically moment-curvature methods based on stress-strain compatibility are employed to determine the resistance of the section at this level of stress. Therefore, this calculation is ideally suited for a computer program and is included in PennDOT's LRFD Prestressed Concrete Girder Design and Rating Computer Program (PSLRFD).

The end moment is calculated by multiplication of prestressing force by the distance measured from the bottom of the beam to the center of gravity of the strand pattern.

C5.9.4

Delete AC5.9.4.2.1 and AC5.9.4.2.2.

C5.9.4.4P

As a contractor redesign, the flexural stresses at the end of a beam may be controlled by use of mild steel reinforcement or debonding for crack control.

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12 in. from the centerline of bearing.

For box beams, solid section properties shall be used to check the stresses at the centerline of bearing and 12 in. from the centerline of bearing towards mid-span.

The allowable range for f'_c , compressive structural design strength of prestressed beam concrete at 28 days, shall be as follows:

$$1.05 f'_{ci} \leq f'_c \leq 1.18 f'_{ci}$$

where:

f'_{ci} = compressive structural design strength of prestressed beam concrete at transfer (ksi)

COMMENTARY

Past experience has shown that in the design of prestressed beams for PennDOT that, if the concrete strength controls the design, it will usually be f'_{ci} instead of f'_c . Therefore, f'_c is based on the f'_{ci} value. In 1994, the prestressed industry recommended a maximum f'_{ci} equal to 6.8 ksi based on an 8 ksi f'_c . The maximum value of f'_{ci} should be 8.5 ksi based on the maximum f'_c of 10 ksi.

Table 5.9.4.4P-1 - Summary of Allowable Concrete Stresses for Pretensioned Beams

TYPE OF STRESS			PRETENSIONED BEAMS					
			I-BEAMS		BOX BEAMS BASED ON HOLLOW SECTIONS		PLANK BEAMS	
			C.L. BRG.	All Other Locations	C.L. BRG.	All Other Locations	C.L. BRG.	All Other Locations
STRAIGHT STRANDS DESIGN	INITIAL STRESS AT TRANSFER OF P/S	TENSION	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$
		COMPRESSION	$0.6 f'_{ci}$	$0.6 f'_{ci}$	$0.6 f'_{ci}$	$0.6 f'_{ci}$	$0.6 f'_{ci}$	$0.6 f'_{ci}$
	FINAL STRESS UNDER DESIGN DL+P/S	COMPRESSION	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$
	FINAL STRESS UNDER DESIGN LOADS	TENSION**	N/A	$0.0948\sqrt{f'_c}$	N/A	$0.0948\sqrt{f'_c}$	N/A	$0.0948\sqrt{f'_c}$
		COMPRESSION	$0.6 f'_c$	$0.6 f'_c$	$0.6 f'_c$	$0.6 f'_c$	$0.6 f'_c$	$0.6 f'_c$
	FINAL STRESS: COMPRESSION - LIVE LOAD PLUS ONE-HALF OF (DL+P/S)		$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$
DEBONDED* AND/OR DRAPED STRANDS DESIGN	INITIAL STRESS AT TRANSFER OF P/S	TENSION	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$	$0.0948\sqrt{f'_{ci}}$		
		COMPRESSION	$0.6 f'_{ci}$	$0.6 f'_{ci}$	$0.6 f'_{ci}$	$0.6 f'_{ci}$		
	FINAL STRESS UNDER DESIGN DL+P/S	COMPRESSION	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$		
	FINAL STRESS UNDER DESIGN LOADS	TENSION**	N/A	$0.0948\sqrt{f'_c}$	N/A	$0.0948\sqrt{f'_c}$		
		COMPRESSION	$0.6 f'_c$	$0.6 f'_c$	$0.6 f'_c$	$0.6 f'_c$		
	FINAL STRESS: COMPRESSION - LIVE LOAD PLUS ONE-HALF OF (DL+P/S)		$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$	$0.4 f'_c$		

* For requirements of debonding and crack control debonding see D5.11.4.3 and D5.11.4.3.1P.

** The allowable stress for final stress under loads, including creep and shrinkage effect for continuous spans, shall be taken as 0.80 of the modulus of rupture (see D5.4.2.6)

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5.9.4.5P Allowable Concrete Stresses for Segmentally Constructed Bridges*5.9.4.5.1P For Temporary Stresses Before Losses - Fully Prestressed Components**5.9.4.5.1aP Compressive Stresses*

The following limits shall apply:

- in pretensioned components $0.60 f'_{ci}$ (ksi)
- in post-tensioned components $0.55 f'_{ci}$ (ksi)

5.9.4.5.1bP Tensile Stresses

The limits in Table 5.9.4.5.1bP-1 shall apply for tensile stresses.

Table 5.9.4.5.1bP-1 - Temporary Tensile Stress Limits for Segmentally Constructed Bridges Before Losses, Fully Prestressed Components

Location	Stress Limit
Longitudinal Stresses Through Joints in the Precompressed Tensile Zone • joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of $0.5 f_y$; with internal tendons • joints without the minimum bonded auxiliary reinforcement through the joints; with internal tendons • joints with external tendons	$0.0948\sqrt{f'_c}$ maximum tension (ksi) 0.025 ksi minimum compression 0.2 ksi minimum compression
Transverse Stresses Through Joints • For any type of joint	$0.0948\sqrt{f'_c}$ (ksi)
Stresses in Other Areas • For areas without bonded nonprestressed reinforcement • For areas with bonded reinforcement sufficient to carry the calculated tensile force in the concrete computed assuming an uncracked section where the reinforcement is proportioned using a stress of $0.5 f_y$, not to exceed 30 ksi.	0.025 ksi minimum compression $0.19\sqrt{f'_c}$ (ksi)

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*5.9.4.5.2P For Stresses at Service Limit State After Losses - Fully Prestressed Components**5.9.4.5.2aP Compressive Stresses*

Compression shall be investigated using the Service Limit State Load Combination I specified in Table A3.4.1-1, and shall be limited to $0.45 f'_c$ (ksi).

5.9.4.5.2bP Tensile Stresses

For the service load contributions, which involves traffic loading, tension stresses in members with bonded prestressing tendons should be investigated using Load Combination Service III in Table A3.4.1-1.

The limits in Table 5.9.4.5.2bP-1 shall apply.

Table 5.9.4.5.2bP-1 - Temporary Tensile Stress Limits for Segmentally Constructed Bridges at Service Limit State After Losses, Fully Prestressed Components

Location	Stress Limit
Longitudinal Stresses Through Joints in the Precompressed Tensile Zone - Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of $0.5 f_y$; with internal tendons - Joints without the minimum bonded auxiliary reinforcement through the joints; with internal tendons - Joints with external tendons	$0.0948\sqrt{f'_c}$ (ksi) 0.025 ksi minimum compression 0.2 ksi minimum compression
Transverse Stresses Through Joints Tension in the transverse direction in precompressed tensile zone	$0.0948\sqrt{f'_c}$ (ksi)
Stresses in Other Areas - For areas without bonded reinforcement - For areas with bonded reinforcement sufficient to carry the calculated tensile force in the concrete computed assuming an uncracked section where the reinforcement is proportioned using a stress of $0.5 f_y$, not to exceed 30 ksi.	0.025 ksi minimum compression $0.19\sqrt{f'_c}$ (ksi)

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5.9.4.6P Allowable Concrete Stresses for Prestressed Piles*5.9.4.6.1P Compressive Stresses*

The temporary compressive stress before losses in prestressed piles shall be less than $0.6 f'_{ci}$ (ksi).

The final compressive stress in prestressed piles shall be less than $0.4 f'_c$ (ksi).

5.9.4.6.2P Tensile Stresses

The temporary tensile stress before losses in prestressed piles due to handling loads shall be less than $0.158\sqrt{f'_{ci}}$ (ksi).

The final tensile stresses in prestressed piles shall be less than $0.19\sqrt{f'_c}$ (ksi).

C5.9.4.6.1P

For use of prestressed piles, see D5.13.4.4.

5.9.4.7P Partially Prestressed Components

Compression stresses shall be limited as specified in D5.9.4.4P and D5.9.4.5P for fully prestressed components.

Cracking in the precompressed tensile zone may be permitted. The design of partially prestressed members should be based on a cracked section analysis with various service limit states being satisfied. Tensile stress in reinforcement at the service limit state shall be as specified in A5.7.3.4, in which case f_{sa} shall be interpreted as the change in stress after decompression.

5.9.4.8P Tolerances in Prestressed Allowable Stresses

"Tolerance" in this section means an "allowable stress overrun" to avoid unnecessary recycling of computations. The establishment of tolerances does not mean that the allowable design stresses are increased by the tolerance amount. The tolerance is the maximum value for an incidental stress overrun which may occur during the normal design process.

At the temporary stage before all losses (at detensioning), the stress tolerance is 0.025 ksi in tension and 0.050 ksi in compression. At the final stage after losses, the maximum tolerance is 2.5% of the allowable stress.

5.9.5 Loss of Prestress**5.9.5.0P Calculation of Loss of Prestress**

For PennDOT projects, the calculation of loss of prestress shall be accomplished by using A5.9.5 and D5.9.5 as appropriate. The time-dependent losses shall be calculated using refined estimates given in A5.9.5.4 and D5.9.5.4. The approximate lump sum estimate of time-dependent losses given in A5.9.5.3 shall not be used.

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5.9.5.1 Total Loss of Prestress

The following shall replace the second paragraph of A5.9.5.1.

In lieu of more detailed analysis, prestress losses in members constructed and prestressed in a single stage may be taken as:

- In pretensioned members

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \quad (5.9.5.1-1)$$

- In post-tensioned members

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} \quad (5.9.5.1-2)$$

where

Δf_{pT} = total loss (ksi)

Δf_{pF} = loss due to friction (ksi)

Δf_{pA} = loss due to anchorage set (ksi)

Δf_{pES} = loss due to elastic shortening (ksi)

Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi)

The following shall supplement A5.9.5.1.

In pretensioned members, the part of the loss due to relaxation which occurs before transfer may be deducted from the total relaxation.

The total prestress loss shall not be less than 20% of the jacking force.

5.9.5.2 Instantaneous Losses

5.9.5.2.3 Elastic Shortening

5.9.5.2.3a Pretensioned Members

The following shall supplement A5.9.5.2.3a.

The term f_{cgp} , used in Equation A5.9.5.2.3a-1, shall be determined at the mid-span of beam as follows:

$$f_{cgp} = \frac{\frac{A_{ps}}{A_{beam}} f_s 2 \left(1 + \frac{e^2 A_{beam}}{I_g} \right) - \frac{M_{DLb} e}{I_g}}{1 + \frac{A_{ps}}{A_{beam}} \left(\frac{E_p}{E_{ci}} \right) \left(1 + \frac{e^2 A_{beam}}{I_g} \right)} \quad (5.9.5.2.3a-2P)$$

COMMENTARY

C5.9.5.1

The following shall be added to the end of the first sentence of AC5.9.5.1.

...and approved by the Chief Bridge Engineer.

C5.9.5.2.3a

The following shall supplement AC5.9.5.2.3a.

In determining f_{cgp} , the LRFD Specification allows the use of an approximate value of the stress in the strands immediately after transfer, f_{pi} , of $0.70 f_{pu}$. A more rigorous calculation of f_{pi} , however, can be made by taking the jacking stress, f_{pj} , and subtracting the initial losses, Δf_{pES} . However, f_{pi} is a function of Δf_{pES} which contains f_{cgp} . Thus, Equation 5.9.5.2.3a-2P was developed as a closed form solution for f_{cgp} based on the following three equations.

$$f_{cgp} = \frac{f_{pi} A_{ps}}{A_{beam}} + \frac{f_{pi} A_{ps} e^2}{I_g} - \frac{M_{DLb} e}{I_g} \quad (C5.9.5.2.3a-2P)$$

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for which:

$$f_{s2} = f_{pj} \quad (5.9.5.2.3a-3P)$$

where:

- A_{beam} = basic beam cross-sectional area (in.²)
- A_{ps} = area of prestressing strand (in.²)
- E_{ci} = modulus of elasticity of concrete at transfer (ksi)
- E_p = modulus of elasticity of prestressing strand (ksi)
- e = the eccentricity of prestressing strand to the centroid of beam (in.)
- f_{pj} = stress in the prestressing steel at jacking (ksi)
- I_g = moment of inertia of basic beam (in.⁴)
- M_{DLb} = moment due to dead load of beam including interior diaphragms (kip-in.)

5.9.5.2.4P Prestress Stress at Transfer

The prestress stress immediately following transfer shall be taken as:

$$f_{pt} = f_{pj} - \Delta f_{pES}$$

where:

- f_{pj} = stress in the prestressing steel at jacking (ksi)
- Δf_{pES} = loss due to elastic shortening (ksi)

5.9.5.3 Approximate Estimate of Time-Dependent Losses

Delete A5.9.5.3.

5.9.5.4 Refined Estimates of Time-Dependent Losses

5.9.5.4.3 Losses: Time of Deck Placement to Final Time

5.9.5.4.3c Relaxation of Prestressing Strands

The following shall supplement A5.9.5.4.3c.

The total loss due to relaxation should be based on test data which has been approved by the Chief Bridge Engineer. If test data is not available, the total loss shall be assumed to be 3.0 ksi.

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where:

$$f_{pi} = f_{pj} - \Delta f_{pES} = f_{s2} - \Delta f_{pES} \quad (C5.9.5.2.3a-3P)$$

for which:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \quad (C5.9.5.2.3a-4P)$$

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5.10 DETAILS OF REINFORCEMENT**5.10.2 Hooks and Bends****5.10.2.1 Standard Hooks**

The following shall supplement A5.10.2.1.

Reinforcement bars shall not be provided with hooks unless required by design or as detailed in the Standard Drawings; however, dowels anchored into footings of substructures that carry primary stresses shall be provided with hooks.

5.10.2.3 Minimum Bend Diameters

The following shall replace A5.10.2.3.

Refer to Standard Drawing BC-736M for minimum bend diameter.

5.10.3 Spacing of Reinforcement**5.10.3.1 Minimum Spacing of Reinforcing Bars***5.10.3.1.1 Cast-in-Place Concrete*

The following shall replace the third bulleted item in A5.10.3.1.1.

- 2 1/2 in.

5.10.3.1.2 Precast Concrete

The following shall replace the third bulleted item in A5.10.3.1.2.

- 1 1/2 in.

5.10.3.1.3 Multi-Layers

The following shall replace A5.10.3.1.3.

Except in decks, where parallel reinforcing is placed in two or more layers, with clear distance between layers not exceeding 6 in., the bars in the upper layers shall be placed directly above those in the bottom layer, and the clear distance between layers shall not be less than either 1 1/2 in. or the nominal diameter of the bars.

5.10.3.1.5 Bundled Bars

The following shall supplement A5.10.3.1.5.

Bundled bars shall be tied, wired, or otherwise fastened together to ensure that they remain in their relative position, regardless of their inclination.

C5.10.3.1.5

Delete AC5.10.3.1.5.

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5.10.3.2 Maximum Spacing of Reinforcing Bars

The following shall supplement A5.10.3.2.

For box culverts, retaining walls, abutments, wall-type piers and similar structures, the spacing of the reinforcement shall not exceed either 1.5 times the structural thickness of the member or 24 in..

5.10.3.3 Minimum Spacing of Prestressing Tendons and Ducts*5.10.3.3.1 Pretensioning Strand*

The following shall replace A5.10.3.3.1.

The minimum clear spacing between pretensioning strands shall be the larger of:

- center-to-center spacing of 2 in. or
- clear distance of 2 times the maximum size of aggregate.

The clear distance between strands at the end of a member may be decreased, if justified by performance tests of full-scale prototypes of the design and approved by the Chief Bridge Engineer.

The minimum clear distance between groups of bundled strands shall not be less than either 2 times the maximum size of the aggregate or 2 in..

Pretensioning strands may be bundled to touch one another in an essentially vertical plane at, and between, hold-down devices, provided that the spacing, specified herein, is maintained between individual strands near the ends of the beams for a distance not less than the maximum shielded length plus development length.

Groups of eight strands of 0.52 in. or 0.60 in. diameter or smaller may be bundled linearly to touch one another in a vertical plane at and between hold-down devices. The number of strands bundled in any other manner shall not exceed four.

5.10.3.3.2 Post-Tensioning Ducts - Girders Straight in Plan

The following shall replace A5.10.3.3.2.

Unless otherwise specified herein, the clear distance between straight post-tensioning ducts shall not be less than 2 in. or 2 times the maximum size of the coarse aggregate.

For precast segmental construction when post-tensioning tendons extend through an epoxy joint between components, the clear spacing between post-tensioning ducts shall not be less than the greater of the duct internal diameter or 4.0 in.

Ducts may be bundled together in groups not exceeding three, provided that the spacing, as specified between individual ducts, is maintained between each duct in the zone within 3 ft. of anchorages.

C5.10.3.3.1

The following shall replace AC5.10.3.3.1.

For 0.52 in. diameter strands, a clear distance of 1.48 in. should be considered sufficient to satisfy this requirement.

For 0.60 in. diameter strands, a clear distance of 1.40 in. should be considered sufficient to satisfy this requirement.

When required or permitted by the Chief Bridge Engineer, groups of more than eight strands of 0.52 in. or 0.60 in. diameter or smaller may be bundled to touch one another in a vertical plane.

C5.10.3.3.2

The following shall replace AC5.10.3.3.2.

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For groups of bundled ducts in construction other than segmental, the minimum clear horizontal distance between adjacent bundles shall not be less than 4 in.. For groups of ducts where the ducts are located in two or more horizontal planes, a bundle shall contain no more than two ducts in the same horizontal plane.

The minimum vertical clear distance between bundles shall not be less than 2 in. or 2 times the maximum size of coarse aggregate.

For precast construction, the minimum clear horizontal distance between groups of ducts may be reduced to 3 in..

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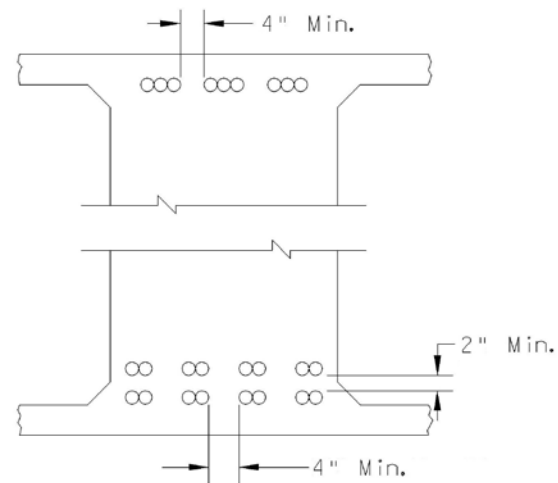


Figure C5.10.3.3.2-1 - Examples of Acceptable Arrangements for Ducts Not Curved in the Horizontal Plane

5.10.6 Transverse Reinforcement for Compression Members

5.10.6.1 General

The following shall supplement A5.10.6.1.

Nominal spiral reinforcement in place of tie reinforcement of No. 4 bars at approximately 12 in. pitch, but not less than the size and spacing shown on the drawings, may be substituted by the contractor at no additional cost to the Department.

5.10.6.2 Spirals

The following shall replace the fifth paragraph of A5.10.6.2.

Splices in spiral reinforcement shall be designed as follows:

The total spiral length may be divided into separate segments. Each segment shall be provided with 1 1/2 extra turns and 135 degree hook at each end. Each hook shall engage a primary reinforcing bar. The maximum distance between spiral segments is limited to the pitch of the spiral.

5.10.6.3 Ties

The following shall replace the fourth paragraph of A5.10.6.3.

Ties shall be arranged so that:

- All longitudinal bars at the corners of a column shall be laterally supported by conforming ties
- The center-to-center distance between any longitudinal bar to the nearest laterally supported bars on either side, along the tie, shall not exceed 24.0 in.

C5.10.6.2P

Allowing the spiral to be divided into segments provides easier constructability and allows the use of hooks at the ends of the spirals. Spirals cannot be spliced with hooks the conventional way, since the hooks will not allow the spliced segments to be threaded together.

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Where the column design is based on plastic hinging capability, no longitudinal bar shall be farther than 6.0 in. clear on each side along the tie from such a laterally supported bar. Where the bars are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

5.10.8 Shrinkage and Temperature Reinforcement

The following shall replace the first bullet of A5.10.8.

- 1.5 times the component structural thickness or 18 in.

The following shall supplement A5.10.8.

For concrete members which have a sacrificial wearing surface or architectural treatments, the total thickness of the member shall be used to determine the area of reinforcement, not the structural thickness.

For components which support load, the minimum reinforcement must be the greater of the shrinkage and temperature reinforcement requirements in A5.10.8, the minimum reinforcement requirements in D5.10.8.1P or as follows:

The total area of reinforcement provided shall be at least 0.120 in^2 per foot, in each direction and on each exposed face, at a maximum spacing of 18 in.

5.10.8.1P Minimum Reinforcement

Any member subject to loading or stress shall have minimum steel reinforcement of No. 4 bars at 12 in. or No. 5 bars at 18 in.. For ties in reinforced concrete I-posts for sound barrier walls, the minimum steel reinforcing shall be No. 3 bars at 9 in.. Any exception to these criteria must be approved by the Chief Bridge Engineer.

5.10.9 Post-Tensioned Anchorage Zones**5.10.9.2 General Zone and Local Zone***5.10.9.2.3 Local Zone*

The following shall replace the first paragraph of A5.10.9.2.3.

Design of local zones shall either comply with requirements of A5.10.9.7 and D5.10.9.7 or based on the results of acceptance tests as specified in D5.10.9.7.3 and Publication 408.

5.10.9.2.4 Responsibilities

Delete the last sentence of the first paragraph of A5.10.9.2.4.

The following shall replace the second and third paragraphs of A5.10.9.2.4.

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The anchorage device supplier shall be responsible for furnishing anchorage devices which satisfy the requirements of Publication 408. If special anchorage devices are used, the anchorage device supplier shall be responsible for furnishing anchorage devices that also satisfy the acceptance test requirements of D5.10.9.7.3.

5.10.9.3 Design of the General Zone*5.10.9.3.3 Special Anchorage Devices*

The following shall replace A5.10.9.3.3.
The provisions of D5.10.9.7.3 apply.

5.10.9.4 Application of the Strut-and-Tie Model to the Design of General Zone*5.10.9.4.2 Nodes*

The following shall replace the first sentence of A5.10.9.4.2.

Local zones which satisfy the requirements of A5.10.9.7 and D5.10.9.7 may be considered as properly detailed and are adequate nodes.

5.10.9.7 Design of Local Zones*5.10.9.7.2 Bearing Resistance**C5.10.9.4.2*

The following shall be added to the end of the last sentence of the first paragraph of AC5.10.9.4.2.
...and approved by Chief Bridge Engineer.

C5.10.9.7.2

The following shall replace the first paragraph of AC5.10.9.7.2.

These specifications provide bearing pressure limits for anchorage devices, called normal anchorage devices. Alternatively, these limits may be exceeded if an anchorage system passes the acceptance test and is approved by the Chief Bridge Engineer.

*5.10.9.7.3 Special Anchorage Devices**C5.10.9.7.3*

The following shall replace A5.10.9.7.3.

Special anchorage devices that do not satisfy the requirements, specified in A5.10.9.7.2, may be used, provided that they have been tested by an independent testing agency acceptable to the Chief Bridge Engineer, meet the requirements of Publication 408, Section 1108, and are approved by the Chief Bridge Engineer.

The following shall be added to the end of the last sentence of the first paragraph of AC5.10.9.7.3.
...and approved by Chief Bridge Engineer.

5.10.10 Pretensioned Anchorage Zones**5.10.10.1 Splitting Resistance****C5.10.10.1**

The following shall replace A5.10.10.1.

The splitting resistance of pretensioned anchorage zones provided by vertical reinforcement in the ends of pretensioned beams at the service limit state shall be taken as:

The following shall replace AC5.10.10.1

The Department allows crack control debonding as specified in Pub. 408 and had successfully controlled end zone cracking using these provisions.

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$$P_r = f_s A_s$$

where:

- f_s = stress in steel not exceeding 24 ksi
 A_s = total area of vertical reinforcement placed near the ends of the beam at maximum spacing of 3 in.

End blocks shall be investigated to help in reducing splitting stresses for prestressed beams or pier caps with forces in excess of 1800 kips. Closely spaced grids for members with forces in excess of 1800 kips shall have the grid anchored. The reinforcement for the end blocks shall be shown on the shop drawings and shall be in accordance with recommendations of the anchorage fabricator.

5.10.10.2 Confinement Reinforcement

The following shall supplement A5.10.10.2.

For prestressed beams, additional confinement reinforcement shall extend from each end of the beam for 1/3 of the span length.

The additional confinement reinforcement shall not be less than No. 4 deformed bars and match with vertical stirrups with maximum spacing of 21 in..

5.10.11 Provisions for Seismic Design**5.10.11.1 General**

The following shall supplement A5.10.11.1.

Slenderness effects (A5.7.4.3) shall be considered in the design of columns for structures with Site Class E or F soils.

For detailing of piers, see standard drawing BD-629M.

5.10.11.2 Seismic Zone 1

The following shall supplement A5.10.11.2.

The reinforcement requirements as specified in

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C5.10.10.2P

The debonding length for beams without debonded design is equal to the maximum crack controlled debonded length as specified Publication 408, Section 1107.01.

C5.10.11.1

The following shall supplement AC5.10.11.1.

Feedback has been received on structures constructed utilizing seismic details which were difficult to construct. Examples of problematic details include:

- Designers must consider that the reinforcement cage for pier caps are constructed on the ground and placed in one piece. It is exceedingly difficult to erect the reinforcement cage when the bottom longitudinal bars (of the cap) interfere with the column spiral or tie reinforcement. Whenever possible, pier caps should be of sufficient depth that column reinforcement extending into the cap can be fully developed without hooks.
- Some designs are using tie bars with 180° hooks on both ends of the bar. Please note Figure AC5.10.11.4.1d-4 allows for tie bars with a 90° hook on one end.

An alternate tie detail permitted on past projects, to allow for access in the pier shaft reinforcement cage, has been two hook bars with a 180° hook on one end, that have a Class C – Top Bar lap splice.

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D5.10.11.4.1f and D5.10.11.4.5P shall be incorporated on all structures in Pennsylvania. The spacing of the transverse reinforcement over the length of the column shall be as per A5.10.6 and D5.10.6.

For multi-span bridges with site soils classified as Site Class E or F, the transverse reinforcement at the top and bottom of a column shall be specified in A5.10.11.4.1d and A5.10.11.4.1e, regardless of the response acceleration coefficient value.

5.10.11.3 Seismic Zone 2

The following shall supplement A5.10.11.3.

The reinforcement requirements as specified in Sections D5.10.11.4.1f and D5.10.11.4.5P shall also be incorporated.

The spacing of the transverse reinforcement over the length of the splice shall not exceed 6 in. or one-quarter of the minimum member dimension.

5.10.11.4 Seismic Zones 3 and 4*5.10.11.4.1 Column Requirements**5.10.11.4.1c Column Shear and Transverse Reinforcement**C5.10.11.4.1c*

The following shall supplement AC5.10.11.4.1c and is commentary regarding the second bullet of A5.10.11.4.1c.

A5.10.11.4.1 provides provisions on what is considered a pier and a column in regards to seismic design. In most cases, wall type piers will qualify as "piers" for which A5.10.11.4.2 will apply.

*5.10.11.4.1d Transverse Reinforcement for Confinement at Plastic Hinges**C5.10.11.4.1d*

The following shall replace the second paragraph of A5.10.11.4.1d

For a circular column, the volumetric ratio of spiral or circular hoop reinforcement, ρ_s , shall not be less than either that required in A5.7.4.6 or:

$$\rho_s = 0.12 \frac{f'_c}{f_y} \quad (5.10.11.4.1d-1)$$

where:

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

f_y = yield strength of reinforcing bars (ksi)

The following shall supplement A5.10.11.4.1d.

If spirals are used, a 135 degree hook shall be used at each end of the spiral.

The following shall supplement AC5.10.11.4.1d.

This specification ensures sufficient ductility be available in order to utilize the response modification factors. If proper detailing is not provided, ductility is considered inadequate and the response modification factors are not applicable.

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5.10.11.4.1e Spacing of Transverse Reinforcement for Confinement

The following shall replace the last bullet of A5.10.11.4.1e.

- Spaced not to exceed one-quarter of the minimum member dimension, 6.0 times the diameter of the longitudinal reinforcement or 6 in. center-to-center.

5.10.11.4.1f Splices

The following shall replace A5.10.11.4.1f.

The provisions of A5.11.5 shall apply for the design of splices.

Lap splices in longitudinal reinforcement for columns and wall piers with shaft height less than or equal to 50 ft. shall be used only within the center half of the shaft height. For columns and wall piers greater than 50 ft., lap splices are not permitted within a distance not less than the smaller of 1/4 of the column shaft height and 20 ft. at the top and bottom of the column shaft. The splice length shall not be less than 16 in. or 60 bar diameters.

Full-mechanical connection splices conforming to A5.11.5 may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 24 in. measured along the longitudinal axis of the column.

For columns and wall piers, bars which extend 25 ft or less from the top of footing should not be spliced.

Splices shall be staggered such that no more than 50% of the bars are spliced at one location.

5.10.11.4.5P Footing Requirements

The top and bottom mats of reinforcement shall be tied together by a grid of No. 4 tie bars at a maximum spacing of 48 in. in both directions. Ties shall have a 135 degree hook on one end and a 90 degree hook on the opposite end, alternately placed. See Figure DC5.10.11.4.5P-1.

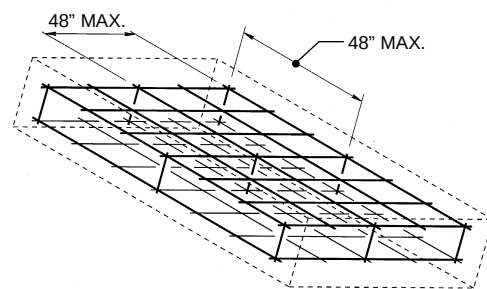
C5.10.11.4.5P

Figure C5.10.11.4.5P-1 Footing Ties

5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11.1 General

5.11.1.2 Flexural Reinforcement

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5.11.1.2.1 *General*

C5.11.1.2.1

The following shall supplement A5.11.1.2.1.

For pier caps of hammerhead piers, the negative moment reinforcement shall be extended for the full-length of the cap.

The following shall supplement AC5.11.1.2.1

For pile-supported footings, hooks shall be provided at both ends of the transverse flexural reinforcement contained in the bottom mat.

Hooks will provide supplementary anchorage.

For a pile-supported footing, consisting of four piles, hooks shall be provided at both ends of the longitudinal and transverse flexural reinforcement contained in the bottom mat.

5.11.2 Development of Reinforcement**5.11.2.1 Deformed Bars and Deformed Wires in Tension**5.11.2.1.1 *Tension Development Length*

The following shall supplement A5.11.2.1.1.

Some engineers are misinterpreting A5.11.2.1.3 by assuming that the modification factors of A5.11.2.1.3 (decreasing ℓ_d) cannot be applied if the basic development length is not modified by A5.11.2.1.2 (increasing ℓ_d).

All applicable modification factors of A5.11.2.1.3 (decreasing ℓ_d) shall be applied to the basic development length, regardless of whether the factors of A5.11.2.1.2 (increasing ℓ_d) have been applied or not.

5.11.2.4 Standard Hooks in Tension5.11.2.4.3 *Hooked Bar Tie Requirements*

The following shall replace the text in A5.11.2.4.3. (Figure A5.11.2.4.3-1 is acceptable and does not need to be replaced.)

For bars being developed by a standard hook at discontinuous ends of members with both side cover and top or bottom cover less than 2 1/2 in., hooked bar shall be enclosed within ties or stirrups spaced, not greater than 3 d_b , along the full development length, ℓ_{dh} , as shown in Figure A5.11.2.4.3-1. In determining the ℓ_{dh} for use in Figure A5.11.2.4.3-1, the third bulleted factor in A5.11.2.4.2 shall not apply.

5.11.3 Development by Mechanical Anchorages

The following shall be added to the end of the first paragraph of A5.11.3.

Mechanical anchorages shall be approved by the Chief Bridge Engineer before installation.

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5.11.4 Development of Prestressing Strand**5.11.4.3 Partially Debonded Strands**

The following shall replace A5.11.4.3.

For debonded strands, the development length calculated in A5.11.4.2, using a value of $\kappa = 2.0$ shall begin at a point where debonding stops and the bonding begins (i.e., not at the end of strand).

The length of debonding of any strand shall be such that all limit states are satisfied, with consideration of the total development resistance at any section being investigated.

The number of partially debonded strands in lieu of draping shall not be greater than 25% of the total number of strands.

The maximum number of debonded strands in a row shall not exceed 50%. The number of debonded strands may be rounded to the next higher number for the case of an odd number of strands in a row; however, ensure the debonding pattern is symmetrical about the vertical centerline of the beam.

Debonded strands are permitted in the bottom row. Debonding the exterior strands of any row in the bottom flange shall not be permitted.

When several strands are debonded in lieu of draping, it may be necessary to have more than one cut-off point, but the number of cut-off points shall be limited to a maximum of six. The design shall provide for a 12 in. minimum distance between each cut-off length.

The number of debonded strands at a cut-off section shall be limited to a maximum of six strands.

Select debonded strand pattern uniformly to avoid stress concentrations.

The cutoff pattern shall provide for an increase in eccentricity at each cut-off.

Debonding of adjacent strand in the same row and/or column shall be avoided. In the webs of box beams, debonded strands shall not occur in consecutive rows. In the web of I-beams, do not debond two strands in consecutive rows. Shop drawings must be checked to comply with this requirement.

5.11.4.3.1P Crack Control Debonding

Debonding is also used as a means of controlling detensioning cracks at the ends of beams. This secondary use of debonding is generally referred to as crack control debonding. The actual practice of crack control debonding is given in Publication 408, Section 1107.01.

The percentage of partially debonded strands in lieu of draping, plus the percentage of crack control debonding, shall not exceed 50%.

C5.11.4.3

The following shall replace AC5.11.4.3.

The limit of six cut-off locations is to accommodate a maximum 25% partially debonded strands. This limit does not include crack control debonding.

SPECIFICATIONS

COMMENTARY

5.11.5 Splices of Bar Reinforcement**5.11.5.2 General Requirements***5.11.5.2.2 Mechanical Connections**C5.11.5.2.2*

The following shall replace the last sentence of the second paragraph of AC5.11.5.2.2.

Only Department pre-approved mechanical connectors shall be permitted.

5.12 DURABILITY**5.12.3 Concrete Cover****C5.12.3**

The following shall replace A5.12.3.

The following minimum concrete cover shall be provided for reinforcement:

Delete the first paragraph of AC5.12.3.

- Concrete cast against and permanently exposed to earth..... 4 in.
- Concrete exposed to earth..... 3 in.
(Except 2 in. minimum may be used for the stem steel of the safety wings and walls supporting barriers as shown in the bridge standards.)
- Concrete exposed to weather and pier columns..... 3 in.
- Concrete deck slab
 - Top reinforcement 2 1/2 in.
 - Bottom reinforcement..... 1 in.
- Concrete not exposed to weather or in contact with ground
 - Primary reinforcement..... 1 1/2 in.
 - Stirrup, tie and spiral 1 in.
- Precast concrete pipes..... see A12.10.4.2.4e
- Prestressed concrete
 - Box beams see BD-661M
 - I-beams see BD-662M
- Reinforced concrete box culverts, cast-in-place
 - Top slab:
 - top bars at grade..... 2 1/2 in.
 - all others 2 in.
 - Bottom slab:
 - top bars 2 1/2 in.
 - bottom bars 3 in.
 - Walls..... 2 in.
- Reinforced concrete box culverts, precast
 - Top slab:
 - top bars at grade..... 2 1/2 in.
 - bottom bars 1 1/2 in.
 - all others 2 in.
 - Bottom slab:
 - top bars 2 in.
 - bottom bars 1 1/2 in.
 - Walls..... 1 1/2 in.

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- Reinforced concrete box culverts, precast with welded deformed wire fabric see A12.11.4.4

5.13 SPECIFIC MEMBERS

5.13.2 Diaphragms, Deep Beams, Brackets, Corbels and Beam Ledges

5.13.2.2 Diaphragms

The following shall replace the fourth paragraph of A5.13.2.2.

For prestressed I-beams and box beams, diaphragm requirements are given on BD-651M.

The following shall supplement A5.13.2.2.

For segmental box girder bridges, location and design of diaphragm shall be approved by the Chief Bridge Engineer.

5.13.3 Footings

5.13.3.4 Moment in Footings

5.13.3.5 Distribution of Moment Reinforcement

COMMENTARY

C5.13.3.4

The following shall supplement AC 5.13.3.4.

For the PAPIER and VBent programs, the design moment will be determined at the 3/4 point of the pressure (for spread footings) or pile load (for pile foundation), to account for the effects of unsymmetrical loading.

C5.13.3.5P

Figure C5.13.3.5P-1 provides a graphical representation of how the reinforcement in the long and short direction should be applied.

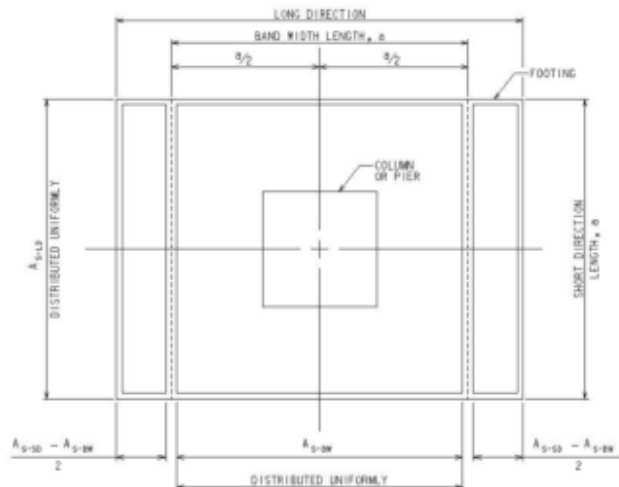


Figure C5.13.3.5P-1 - Example of Distribution of Reinforcement

SPECIFICATIONS

COMMENTARY

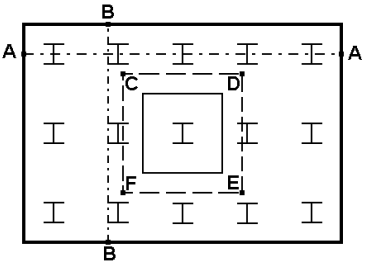
5.13.3.6 Shear in Slabs and Footings

5.13.3.6.1 Critical Sections for Shear

C5.13.3.6.1

The following shall supplement AC5.13.3.6.1 and is commentary regarding the last paragraph of A5.13.3.6.1.

Figure C5.13.3.6.1P-2 provides an example of how to proportion the load from a pile when the pile is intersected by shear section line.



ASSUME ONE FLANGE AREA EQUALS WEB AREA
ASSUME ALL PILES HAVE EQUAL LOAD

SHEAR DESIGN OF THE FOOTING PILE LOADS	
DESIGN SECTION	LOAD
A-A	5 PILES (.5) = 2.5 PILES
B-B	3 PILES
C-D-E-F	12 PILES + 2 ($\frac{5}{8}$) PILES = 13.67 PILES

Figure C5.13.3.6.1P-2 - Example of Portioning Pile Loads for Shear Design

5.13.3.8 Transfer of Force at Base of Column

5.13.3.8.1P Unreinforced Concrete Footings

5.13.3.8.1aP Design Stress

Design stresses in plain concrete footings or pedestals shall be computed assuming a linear stress distribution. For footings and pedestals cast against soil, effective thickness used in computing stresses shall be taken as the overall thickness minus 3 in.. Bending need not be considered, unless projection of footing from face to support member exceeds the footing effective thickness.

5.13.3.8.1bP Pedestals

The ratio of unsupported height to average least lateral dimension of plain concrete pedestals shall not exceed 3.

5.13.4 Concrete Piles

5.13.4.1 General

The following shall supplement A5.13.4.1.

C5.13.4.1

The following shall supplement AC5.13.4.1.

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Piles shall be designed as structural members capable of safely supporting all imposed loads. A pile group composed of both vertical and battered piles which is subjected to lateral load shall be designed assuming that all lateral load is resisted by the horizontal component of the axial capacity of the battered piles. For a pile group composed of only vertical piles which is subjected to lateral load, the pile structural analysis shall include explicit consideration of soil-structure interaction effects using a COM624P (Wang and Reese, 1993) or LPILE 5.0 (ENSOF, Inc. 2004 for LPILE 5.0) Analysis.

5.13.4.4 Precast Prestressed Piles

The following shall supplement A5.13.4.4.

Precast prestressed piles are not to be used unless approved by the Chief Bridge Engineer.

5.13.4.6 Seismic Requirements*5.13.4.6.1 Zone 1*

The following shall replace A5.13.4.6.1.

The requirements for Zone 1 shall be as specified for Zone 2, in A5.13.4.6.2.

5.13.4.7P Structural Resistance*5.13.4.7.1P Concrete Filled Steel Pipe Piles*

The factored axial resistance of undamaged concrete filled steel pipe piles shall be taken as:

$$P_r = \phi_s A_{st} f_y + \phi_c 0.85 A_{gc} f'_c \quad (5.13.4.7.1P-1)$$

where:

ϕ_c = resistance factor for concrete as specified in D5.5.4.2

ϕ_s = resistance factor for steel as specified in D6.5.4.2

A_{st} = area of steel (in.²)

A_{gc} = gross cross-sectional area of concrete (in.²)

5.13.4.7.2P Prestressed Concrete Piles

The factored axial resistance prestressed concrete piles shall be taken as:

$$P_r = \phi (f'_c - 0.87 f_{pe}) A_{gc} \quad (5.13.4.7.2P-1)$$

where:

ϕ = resistance factor as specified in D5.5.4.2

f_{pe} = stress in member due to prestress (ksi)

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Resistance factors, ϕ , for the Strength Limit State shall be taken as specified in D5.5.4.2.

The resistance factors presented in D5.5.4.2 have been selected in a manner such that, when combined with an average load factor of 1.45, the equivalent factor of safety calculated as the ratio of the appropriate load to resistance factors is comparable to the factor of safety previously used by the Department.

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5.13.4.7.3P Precast Concrete Piles

The resistance factor specified in D5.5.4.2 shall be applied for determination of the stress levels in the gross cross-sectional area of the concrete.

5.13.4.7.4P Buckling

Instability of piles which extend through water or air shall be accounted for as specified in A5.7.4.3 using the resistance factors specified in D5.5.4.2. Piles which extend through air or water shall be assumed to be fixed at some depth below the ground. Stability shall be determined in accordance with provisions in A5.7.4.3 for compression members using an equivalent length of the pile equal to the laterally unsupported length, plus an embedded depth to fixity. The depth to fixity shall be determined in accordance with D10.7.3.13.4 or use of COM624P or LPILE 5.0.

5.13.4.7.5P Maximum Permissible Driving Stresses

Maximum permissible driving stresses shall be taken as specified in D10.7.8.

5.14 PROVISIONS FOR STRUCTURE TYPES**5.14.1 Beams and Girders****5.14.1.1 General**

The following shall supplement A5.14.1.1.

Prestressed beam cross-section shall be selected from the Standard Drawing BD-652M. Beam cross-sections and section properties that deviate from the current standards will require specific approval from the Chief Bridge Engineer. In no case shall minimum thickness of beams be less than those given in D5.14.1.2.2.

The allowable skew limitations shown in the Standard Drawing BD-651M shall not be exceeded unless approved by the Chief Bridge Engineer. The limitations shown are maximum limits, tampering with those values may result in severe end zone distress.

5.14.1.2 Precast Beams*5.14.1.2.2 Extreme Dimensions*

The following shall replace A5.14.1.2.2.

The maximum dimensions and weight of precast members manufactured at off-site casting yards shall conform to Department hauling restrictions given in PP1.13.2.

The thickness of any part of precast concrete beams shall not be less than:

C5.14.1.1

The following shall supplement AC5.14.1.1.

Dapping at beam ends, where required, is not considered to be a deviation from the standard sections.

C5.14.1.2.2

Delete AC5.14.1.2.2.

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I-BEAMS

Top Flange.....	5 in.
Webs.....	8 in.

BULB TEES

Top Flange.....	4.5 in.
Webs.....	7 in.

BOX BEAMS

Top Flange.....	5 in.
Top Flange of Composite Box Beams.....	3 in.
Webs.....	5 in.
Bottom Flange	5 1/2 in.

Box beams with internal haunches at the ends may be used in lieu of box beams with draped strands. An internal diaphragm at least 6 in. thick shall be provided at the end of the haunch where the bottom slab thickness changes from 5 1/2 in. to a maximum slab thickness of 8 in. often near the design drape point. A tapered void from the diaphragm to the end block shall be considered which would improve the internal stress flow. The top fiber tensile stresses shall not exceed the values given in Table D5.9.4.4P-1.

Field splices in precast members are not permitted, except if approval is obtained from the Chief Bridge Engineer during TS&L stage.

5.14.1.2.4 Detail Design

The following shall supplement A5.14.1.2.4.

For precast concrete beams used in multi-beam decks, the maximum spacing of the welded shear connectors shall not exceed the lesser of 5 ft. and the width of the flange of the precast member. Welded shear connector anchors shall be located within the middle third of the slab thickness.

5.14.1.2.5 Concrete Strength

Delete A5.14.1.2.5.

C5.14.1.2.5

Delete AC5.14.1.2.5.

5.14.1.3 Spliced Precast Girders*5.14.1.3.1 General*

The following shall replace the first paragraph of A5.14.1.3.1.

This article applies to bridges consisting of precast concrete girders fabricated in segments that are joined or spliced longitudinally to form the girders in the final structure by using a cast-in-place closure placement (splice) at locations other than at the piers. Bridges consisting of precast concrete girders or cast-in-place concrete slabs made continuous for transient loads by using a cast-in-place closure placement at the piers with tensile reinforcement located in the slab are covered in D5.14.1.4.1.

The following shall replace the fourth paragraph of

C5.14.1.3.1

The following shall replace the fourth and fifth bullets in the fourth paragraph of AC5.14.1.3.1.

- The bridge cross-section is comprised of several individual girders with a cast-in-place concrete composite deck rather than precasting the full width and depth of the superstructure as one piece. In some cases, the deck may be divided into pieces that are integrally cast with each girder. A bridge of this type is completed by connecting the girders across the longitudinal joints without a cast-in-place composite deck. The latter form of construction requires approval of the Chief Bridge Engineer prior to the start of the

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A5.14.1.3.1.

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor to the construction method or to the design shall comply with the requirements of A5.14.2.5, D5.14.2.5 and the Department's policies regarding alternative designs.

The following shall replace the seventh, eighth, ninth and tenth paragraphs of A5.14.1.3.1.

Prestress losses in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Articles A5.9.5 and D5.9.5. The effects of combined pretensioning and post-tensioning and staged post-tensioning shall be considered.

When required, the effects of creep and shrinkage in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Articles A5.4.2.3 and D5.4.2.3.

Precast deck girder bridges, for which some or all of the deck is cast integrally with a girder, may be spliced. Spliced structures of this type, which have longitudinal joints in the deck between each deck girder, shall comply with the additional requirements of Articles A5.14.4.3 and D5.14.4.3.

Spliced precast girders may be made continuous for some permanent loads using details for simple span precast girders made continuous. In such cases, design shall conform to the applicable requirements of Articles A5.14.1.4 and D5.14.1.4.

5.14.1.3.2 *Joints between Segments*5.14.1.3.2b *Details of Closure Joints*

The following shall replace the fourth and fifth paragraphs of A5.14.1.3.2b.

If the joint is located in the span, its web transverse reinforcement per unit length of the girder shall be the larger of that in the adjacent girder segments.

The face of the precast segments at closure joints shall have shear keys in accordance with A5.14.2.4.2. Shear keys shall be provided at the ends of the girder segments on either side of the closure joints.

5.14.1.3.2d *Joint Design*

The following shall replace the second paragraph of A5.14.1.3.2d.

Stress limits for concrete stresses in joints at the service limit state after losses specified in D5.9.4.4P for segmentally constructed bridges shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

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design.

- Girder sections are used, such as bulb tee, rather than closed cell boxes with wide monolithic flanges.

The following shall replace the fifth paragraph of AC5.14.1.3.1.

Provisional ducts are required for segmental construction (Articles A5.14.2.3.8a and D5.14.2.3.8a) to provide for possible adjustment of prestress force during construction. Similar requirements are not given for spliced precast girder bridges because of the redundancy provided by a greater number of webs and tendons, and typically lower friction losses because of fewer joint locations.

C5.14.1.3.2b

The following shall replace the last paragraph of AC5.14.1.3.2b.

Roughening the ends of the girder segments on either side of closure joints is not considered sufficient to develop the required shear strength.

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5.14.1.3.3 Girder Segment Design

The following shall replace A5.14.1.3.3.

Stress limits for temporary concrete stresses in girder segments before losses specified in D5.9.4.4P for other than segmentally constructed bridges shall apply at each stage of prestressing (pretensioning or post-tensioning) with due consideration for all applicable loads during construction. The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in girder segments at the service limit state after losses specified in D5.9.4.4P for other than segmentally constructed bridges shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

Where girder segments are precast without prestressed reinforcement, the provisions of Articles A5.7.3.4 and D5.7.3.4 shall apply until post-tensioning is applied.

Where variable depth girder segments are used, the effect of inclined compression shall be considered.

The potential for buckling of tall thin web sections shall be considered.

5.14.1.4 Bridges Composed of Simple Span Precast Girders Made Continuous*5.14.1.4.1 General*

The following shall replace A5.14.1.4.1.

This article applies to bridges consisting of precast concrete girders or cast-in-place concrete slabs made continuous for transient loads by using a cast-in-place closure placement at the piers with tensile reinforcement located in the slab. Bridges made continuous by closure pours (splices) at locations other than at the piers are covered by D5.14.1.3.

All prestressed concrete bridges shall be designed for all applicable limit states as continuous for live load and superimposed dead load with a continuous deck to eliminate joints in the deck slab. The same number of beams shall be used in adjacent spans, unless special approval is granted by the Chief Bridge Engineer at the TS&L stage. The beam depth for box beams and I-beams shall be within 6 in. and for bulb-tee beams shall be within 8 in. for beams in adjacent spans.

An added requirement for prestressed concrete beam bridges made continuous for superimposed dead load and live load is to design all structure components for the more critical condition of full continuity or the complete loss of continuity at the diaphragms over the interior supports. Positive moment steel in the continuity diaphragm(s) is not required since the structure design is not predicated on ensuring continuity.

A full continuity design option (not including the simple span check) may only be used if there is a clear

C5.14.1.4.1

The following shall replace AC5.14.1.4.1.

Some of the instances where the full continuity option should be used are as follows:

- When cost analysis shows simple spans design would place the P/S industry at an economic disadvantage (an extra line of girders is required).
- When longer spans are required for a specific project site.
- When underclearance is a limiting factor in beam selection.

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economic advantage to be gained. For this option, positive moment reinforcement shall be provided in the continuity diaphragm as required by design. Justification for utilizing a full continuity design option must be submitted as part of the TS&L submission.

5.14.1.4.2 Restraint Moments

The following shall supplement A5.14.1.4.2.

When a full continuity design option (positive moment connection) is used per D5.14.1.4.8, restraint moments shall be considered.

5.14.1.4.3 Material Properties

The following shall replace A5.14.1.4.3.

Creep and shrinkage properties of the girder concrete and the shrinkage properties of the deck slab concrete shall be in accordance with the provisions of A5.4.2.3 and D5.4.2.3. The restraint effect of reinforcement on concrete shrinkage may not be considered.

5.14.1.4.4 Age of Girder When Continuity is Established

The following shall replace A5.14.1.4.4.

When a full continuity design option (positive moment connection) is used per D5.14.1.4.8, the minimum age of the precast girder when continuity is established shall be specified on the contract drawings. This age shall be used for calculating restrain moments due to creep and shrinkage.

The minimum age should be 30 days. The creep and shrinkage effects shall be investigated for 30 and 450 days from the day the girders are cast until the deck is poured. The worst effects from either of these two data points shall be used in the positive and negative moment splice designs.

Provide the following note on the contract drawings:

The continuous girder design and the design of both the positive and negative moment connections at the interior supports is based on a minimum girder age of ____ days prior to establishing continuity.

5.14.1.4.5 Degree of Continuity at Various Limit States

Delete A5.14.1.4.5.

5.14.1.4.7 Strength Limit State

The following shall supplement A5.14.1.4.7.

The reinforcement in the deck slab shall satisfy the minimum reinforcement of D5.7.3.3.2 and the crack control requirements of A5.7.3.4 and D5.7.3.4.

5.14.1.4.8 Negative Moment Connection

The following shall replace A5.14.1.4.8.

The negative moment splice of the beams at a pier shall

C5.14.1.4.3

Delete the second paragraph of AC5.14.1.4.3 in its entirety.

C5.14.1.4.5

Delete AC5.14.1.4.5.

C5.14.1.4.8

The following shall replace AC5.14.1.4.8.

If the longitudinal bar size required in the deck over

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be designed for moments from an analysis assuming full continuity for superimposed dead load and live load. Creep and shrinkage effects shall be included when a full continuity option is utilized. When two precast beams of different depth are used in a bridge made continuous for live load, the longitudinal deck mild steel reinforcement for the negative moment splice shall be designed for the smaller section. The longitudinal slab reinforcement steel designed from the above analysis should not be less than that calculated using D6.10.1.7. The reinforcement shall be extended to a minimum distance of 0.15 times the span length or to the superimposed dead load point of contraflexure, plus development length, whichever is greater. Cut-off points for this reinforcement steel shall be staggered (see A5.11.1.2.1).

For spread and adjacent box beams, the longitudinal slab reinforcement required to make the negative moment splice of the beams shall be taken as the larger calculated based on solid and hollow section properties as follows:

- The solid box beam section shall be considered at the negative moment spike at the center of continuity.
- The hollow box beam section shall be considered at the center of the bearing with the corresponding moment.

If a full continuity design is utilized, the positive moment splice of the beam at a pier shall be designed for moments from an analysis assuming full continuity for superimposed dead load, live load and the effects of creep and shrinkage moments.

At interior piers where the diaphragms contain the closure placement, the design may be based on the strength of the concrete in the precast elements.

5.14.1.4.9 Positive Moment Connections

5.14.1.4.9a General

The following shall replace the first paragraph and bulleted list of A5.14.1.4.9a.

Positive moment connections at continuity diaphragms shall be made with mild reinforcement embedded in the precast girders and developed into both the girder and continuity diaphragm.

Delete the second paragraph of A5.14.1.4.9a.

5.14.1.4.9b Positive Moment Connection Using Mild Reinforcement

The following shall supplement A5.14.1.4.9b.

For details of positive moment connections, see Standard Drawings BD-664M and BD-665M.

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adjacent box beams at a pier to resist negative live load moment is greater than a No. 4 bar size, the slab thickness may need to be increased to more than 5 in. to provide 2 1/2 in. minimum concrete cover over the top bar mat.

C5.14.1.4.9a

Delete the first and second paragraphs of AC5.14.1.4.9a.

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*5.14.1.4.9c Positive Moment Connection Using Prestressing Strand**C5.14.1.4.9c*

Delete A5.14.1.4.9c.

Delete AC5.14.1.4.9c.

*5.14.1.4.11P Precast Girder Design**C5.14.1.4.11P*

The girder design shall be based on:

- a simple span analysis for non-composite dead load and
- the more critical of either a continuous span analysis assuming full continuity, or a simple span analysis assuming the complete loss of continuity for composite dead load and live load (without creep and shrinkage effects), or a continuous span analysis assuming full continuity for composite dead load and live load with creep and shrinkage effects if a full continuity design option is approved by the Chief Bridge Engineer

The maximum bar size shall be limited to #8 in top layer and #6 in bottom layer of the deck slab. Minimum spacing shall be as specified in Articles A5.10.3 and D5.10.3.

The span length for the simple span analysis for noncomposite dead load shall be the length between the centerline of bearings. The span length for the continuous span analysis or a simple span analysis of composite dead load and live load shall be the length between the centerline of piers for interior span and between the centerline of pier and centerline of end bearing for exterior spans.

If the girders are designed using the “simple span check option” in PSLRFD, a separate run with the “continuous analysis” option is not required.

For simple span design option, with prior approval from Chief Bridge Engineer, the bottom layer reinforcement bar size may be increased to a #8 bar provided a minimum clear distance of 2 3/4 in. between reinforcement bars is maintained at all splice locations and the requirements of A5.7.3.4 and D5.7.3.4 are met.

5.14.1.4.12P Precast Girder and Deck Placement

The deck pouring sequence for prestressed concrete bridges made continuous for live load is as follows:

1. Place intermediate diaphragms, shear blocks between girders and end diaphragms at abutments.
2. Place slab in positive moment areas.
3. Place continuity diaphragms at piers.
4. Place slab in negative moment areas. This can be placed two hours after the continuity diaphragms are placed, when the diaphragm depth is over 3 ft..
5. Place barriers in positive moment region, then negative moment region, unless continuous placement can be maintained.

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COMMENTARY

5.14.2 Segmental Construction**5.14.2.1 General**

The following shall supplement A5.14.2.1.

The design and construction details of segmental bridges shall be such that:

- deck is replaceable
- only internal bonded post-tensioning is used (except external unbonded post-tensioning may be used for the temporary condition on future strengthening)

5.14.2.3 Design**5.14.2.3.3 Construction Load Combinations at the Service Limit State**

The following table shall replace Table A5.14.2.3.3-1.

C5.14.2.1

The following shall replace the first and second paragraph of AC5.14.2.1.

For segmental construction, superstructures of single or multiple box sections are generally used. Segmental construction includes construction by free cantilever, span-by-span, or incremental launching methods using either precast or cast-in-place concrete segments which are joined to produce either continuous or simple spans.

Bridges utilizing beam type sections may also be constructed using segmental construction techniques. Such bridges, which are referred to as spliced precast girder bridges, are considered as a special case of conventional bridges. The design of such bridges is covered in A5.14.1.3 and D5.14.1.3.

Table 5.14.2.3.3-1 - Load Factors and Allowable Tensile Stresses for Construction Load Combinations

Load Combination	LOAD FACTORS															STRESS LIMITS				See Note (A5.14.2.3.3)
	Dead Load			Live Load			Wind Load			Other Loads						Flexural Tension		Principal Tension		
	DC	DIFF	U	CE CLL	IE	CLE	WS	WUP	WE	CR	SH	TU	TG	EH EV ES	WA	Excluding "Other Loads"	Including "Other Loads"	Excluding "Other Loads"	Including "Other Loads"	
a	1.0	1.0	0.0	1.0	1.0	0.0	0.0	0.0	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.095\sqrt{f'c}$	$0.190\sqrt{f'c}$	$0.055\sqrt{f'c}$	$0.109\sqrt{f'c}$	
b	1.0	0.0	1.0	1.0	1.0	0.0	0.0	0.0	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.095\sqrt{f'c}$	$0.190\sqrt{f'c}$	$0.055\sqrt{f'c}$	$0.109\sqrt{f'c}$	
c	1.0	1.0	0.0	0.0	0.0	0.0	0.7	0.7	0.0	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.095\sqrt{f'c}$	$0.190\sqrt{f'c}$	$0.055\sqrt{f'c}$	$0.109\sqrt{f'c}$	
d	1.0	1.0	0.0	1.0	0.0	0.0	0.7	1.0	0.7	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.095\sqrt{f'c}$	$0.190\sqrt{f'c}$	$0.055\sqrt{f'c}$	$0.109\sqrt{f'c}$	1
e	1.0	0.0	1.0	1.0	1.0	0.0	0.3	0.0	0.3	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.095\sqrt{f'c}$	$0.190\sqrt{f'c}$	$0.055\sqrt{f'c}$	$0.109\sqrt{f'c}$	2
f	1.0	0.0	0.0	1.0	1.0	1.0	0.3	0.0	0.3	1.0	1.0	1.0	γ_{TG}	1.0	1.0	$0.095\sqrt{f'c}$	$0.190\sqrt{f'c}$	$0.055\sqrt{f'c}$	$0.109\sqrt{f'c}$	3

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5.14.2.3.6 Creep and Shrinkage

The following shall replace the first sentence of the first paragraph of A5.14.2.3.6.

Creep coefficient $\Psi(t, t_i)$ shall be determined either in accordance with A5.4.2.3 and D5.4.2.3 or by comprehensive tests which are approved by the Chief Bridge Engineer.

*5.14.2.3.7 Prestress Losses**C5.14.2.3.7*

The following shall supplement AC5.14.2.3.7.

In-place friction test results shall be approved by the Chief Bridge Engineer.

*5.14.2.3.8 Provisional Post-Tensioning Ducts and Anchorages**5.14.2.3.8a General*

The following shall supplement A5.14.2.3.8a.

External ducts are not permitted, except for temporary construction or rehabilitation projects.

*5.14.2.3.10 Box Girder Cross-Section Dimensions and Details**5.14.2.3.10e Overlays*

The following shall replace A5.14.2.3.10e.

Overlays shall be considered for all bridge decks exposed to freeze thaw cycles and application of deicing chemicals. The governing authority should consider providing additional protection against penetration of chlorides. For all types of segmental bridges (precast and cast-in-place), it is recommended that this additional protection be provided by the addition of a minimum of 1.5 in. of concrete cover, added as an overlay. The governing authority may require specific materials and placement techniques stipulated by local practices.

5.14.2.5 Use of Alternative Construction Methods

The following shall supplement A5.14.2.5.

The use of alternative segmental construction methods requires the approval of the Chief Bridge Engineer.

5.14.4 Slab Superstructures**5.14.4.3 Precast Deck Bridges***5.14.4.3.1 General*

Delete the second paragraph of A5.14.4.3.1.

5.14.4.3.3 Shear-Flexure Transfer Joints

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5.14.4.3.3d *Longitudinal Construction Joints*

C5.14.4.3.3d

The following shall replace A5.14.4.3.3d.

Delete AC5.14.4.3.3d

For details of longitudinal construction joints, see standard drawing BC-775M.

5.14.4.3.3f *Structural Overlay*

The following shall replace A5.14.4.3.3f.

When a structural overlay is used to qualify for improved load distribution as provided in A4.6.2.2.2 and A4.6.2.2.3, the thickness of structural concrete overlay shall not be less than 5 in.. An isotropic layer of reinforcement shall be provided in accordance with the requirements of A5.10.8. The top surface of the precast components shall be roughened.

5.14.5 Additional Provisions for Culverts**5.14.5.4P Shear Resistance Provided by Single Bent-Up Bars in Box Culverts**

The additional nominal shear resistance provided by a single bar or single group of parallel bars all bent up at the same distance from the support shall be taken as:

$$V_s = A_v f_y \sin \alpha \leq 0.948 \sqrt{f'_c} b d_v \quad (5.14.5.4-1P)$$

where:

- A_v = cross sectional area of bent-up bar (in.²)
- f_y = yield strength of bent-up bar (ksi)
- α = angle of inclination of transverse reinforcement to longitudinal axis (degrees)
- b = width of concrete section (in.)
- d_v = effective shear depth as determined in A5.8.2.7 (in.)
- f'_c = specified compressive design strength of concrete at 28 days, unless other age is specified (ksi)

5.14.6P Post-Tensioned Pier Caps

For inverted T-post-tensioned pier caps, a detailed stress analysis and detailed design is required for bracket (corbel) reinforcement, including all forces acting on any pedestal.

This investigation must include, but is not limited to, bearing pressure, bending, flexural shear, diagonal tension, pure shear, punching shear, horizontal forces, vertical forces and torsional forces.

Investigate the combined effect of all vertical and horizontal superstructure forces and any other conditions which are warranted.

To compensate for incidental field adjustments in the location of bearings, all pier's columns, solid piers and abutment's stems shall be designed for a 2 in. longitudinal eccentricity off the theoretical centerline of bearing. The

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eccentricity need not be considered for footing design.

End faces shall be proportioned to allow proper placement of anchor plates. Outside edges of anchor plates shall not be less than 3 in. from an exposed edge.

In addition to the other strength and service limit state checks, concrete tensile stresses due to Service III Load Combination loads shall not exceed $0.0948\sqrt{f'_c}$.

5.15 REFERENCES

The following shall supplement A5.15.

Lutz, LeRoy, A., "Crack Control Factor for Bundled Bars and for Bars of Different Sizes", ACI Journal, January 1974, pp 9-10.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 6 – STEEL STRUCTURES

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COMMENTARY

6.1 SCOPE**6.1.1P Restrictions of Steel Bridge Types****6.1.1.1P Steel Tied-Arch Bridges**

Steel tied-arch bridges shall be used only after thorough consideration has been given to all factors in design, fabrication and erection, and if the design is approved by the Chief Bridge Engineer. In the preliminary stage, the tied-arch must show a marked economic advantage over alternate designs to warrant further consideration. Tied-arch structures are currently unacceptable to FHWA. Moreover, refer to the FHWA Technical Advisory T-5140.4, dated September 28, 1978, for the problems pertinent to tied-arch structures.

Transverse welds on the tie girders shall be avoided, where possible. Bolted connections shall be used instead of transverse welds.

On Langer-girder tied-arch bridges (those tied arches where the tie girder acts as the major flexural member in addition to providing horizontal reactions to the arch rib) with box girders functioning as tie girders, the internal diaphragms stiffening the box at the floorbeam connections shall be attached to both flanges, as well as the webs. A tie plate should be placed between the tie-girder flange and the floorbeam flange if they lie essentially in the same plane.

Hangers composed of multiple bridge strands shall have either spacers between the strands or dampers, or both.

The dynamic response of the bridge due to traffic shall be investigated by an appropriate three-dimensional, forced-vibration dynamic analysis, especially for tied-arch bridges that do not employ Langer-girders.

6.1.1.2P Steel Box Bridges

Steel box bridges shall be used only after thorough consideration has been given to all factors in design, fabrication, erection and future in-depth inspection, and if the design is approved by the Chief Bridge Engineer. In the preliminary stage, the steel box design must show a marked economic or aesthetic advantage over alternate designs to warrant further consideration.

C6.1.1.1P

Steel tied-arch bridges have experienced such problems as lamellar tearing in the hanger connections, detrimental vibration in the main structure and cables, and cracking in fracture-critical members. The design, detailing, and fabrication of the floorbeams are critical for long-term performance. Fatigue cracking has occurred in floorbeams due to out-of-plane distortion in combination with abrupt termination of the flange; proper coping and grinding of the cope were not performed.

The designer must use intuitive engineering judgment when selecting the type, location and number of spacers used between the strands of a hanger composed of multiple bridge strands. The need for spacers is not based upon a calculated analysis, but rather on the observation that some bridges without spacers experienced problems and were subsequently retrofitted with spacers.

C6.1.1.2P

Even though steel box girders may provide aesthetically pleasing and sometimes economical structures, the Department has major concerns about steel box girders which are:

- difficult inspection environment,
- inspection complexities,
- future cleaning, painting and/or repair difficulties.
- detailing complexities
- stability during erection

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6.2 DEFINITIONS

The following shall supplement A6.2.

Controlling Flange—top or bottom flange for the smaller section at a point of splice, whichever flange has the maximum elastic flexural stress at its mid-thickness due to the factored loads.

Non-controlling Flange—the flange at a point of splice opposite the controlling flange.

6.3 NOTATION

The following shall supplement A6.3

PTF	=	Pennsylvania Traffic Factor for fatigue analysis (D6.6.1.2.2)
b	=	width of wider flange (in.) (D6.7.7.2)
b_{tf}	=	width of narrower flange (in.) (D6.7.7.2)
D_w	=	clear distance between flanges (in.) (D6.7.7.2)
F_{yw}	=	specified minimum yield strength of a web (ksi) (D6.7.7.2)
R	=	radius of curvature (in.) (D6.7.7.2)
t_w	=	web thickness (in.) (D6.7.7.2)
t_f	=	thickness of wider flange (in.) (D6.7.7.2)
t_{tf}	=	thickness of narrower flange (in.) (D6.7.7.2)

6.4 MATERIALS**6.4.1 Structural Steels**

The following shall supplement A6.4.1.

Poisson's ratio for structural steel shall be assumed to be 0.3 in the elastic range.

Unless directed otherwise, all structural steel shall conform to the specifications for structural steel, ASTM A 709/A 709M, Grade 36. Other types of steel, such as ASTM A 709/A 709M, Grades 50 and 50W, in combination with ASTM A 709/A 709M, Grade 36, or with each other may be considered for economy.

Steel Grades 100 or 100W shall not be used unless written approval has been obtained from the Chief Bridge Engineer.

Unpainted ASTM A 709/A 709M, Grade 50W or HPS-70W, steel shall not be specified without written approval of the Chief Bridge Engineer at the TS&L stage. This policy applies to state and local bridges and bridges where State or Federal funding is utilized. Use in contractor-designed alternates must also be approved at the TS&L stage. Use is not permitted in acidic or corrosive environments, in locations subject to salt water spray or fog, in depressed roadway sections (less than 20 ft. clearance) where salt spray and other pollutants may be trapped, in low underclearance situations where the steel is either less than 5 ft. from normal water elevation or continuously wet, or where the steel may be buried in soil. The use of Grade 50W or HPS-70W steel is not permitted in bridge types where salt spray and dirt accumulation may be a concern (e.g., trusses or inclined-leg bridges) unless corrosion-

C6.4.1

The following shall supplement AC6.4.1.

For additional information on the economics of steel bridges, see PP4.3.

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susceptible regions are painted.

Do not use Grade 50W or HPS-70W steel for expansion dams, or for stringers or other members under open steel decking.

Where the use of Grade 50W or HPS-70W unpainted weathering steel is permitted, the following criteria must be met:

- (a) The number of expansion joints shall be minimized.
- (b) Details to avoid retention of water and debris shall be incorporated in the design.
- (c) The steel shall be painted to a length of at least 1.5 times web depth and a minimum of 5 ft. on each side of the expansion joint.
- (d) Drip plates shall be provided.
- (e) The substructure units shall be protected against staining. Use special drainage details for pier and abutment tops and/or protective coating for reinforced concrete surfaces in accordance with the Publication 408.
- (f) Mechanical fasteners made of AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490), Type 3, weathering steels and stainless steels are suitable for weathering steel bridges. Do not use zinc and cadmium galvanized carbon-steel bolts for weathering steel bridges.
- (g) Load indicator washers are not recommended.

For existing bridges, where Grade 50W unpainted steel is used, clean and paint the beam ends up to 5 ft. from leaking joints, or to where the weathering steel area is exposed to or subject to salt water spray.

6.4.3 Bolts, Nuts and Washers

6.4.3.1 Bolts

The following shall replace A6.4.3.1.

Bolts used as structural fasteners shall conform to one of the following:

- the Standard Specification for Carbon Steel Bolts and Studs, 60 ksi Tensile Strength, ASTM A 307 Grade A or B
- the Standard Specification for Structural Bolts, Steel, Heat-Treated, 120/105 ksi Minimum Tensile Strength with a required minimum tensile strength

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For additional information, refer to NCHRP Report No. 314, Guidelines for the Use of Weathering Steel in Bridges.

Drip bars attached as indicated on BC-753M.

Preferably for weathering steel bridges, use mechanical fasteners made of weathering steel. When stainless steel mechanical fasteners are used with weathering steel bridges, there is a possibility of galvanic corrosion of the weathering steel. Due to the small area of the bolt in relation to the material being bolted, the effect is usually negligible.

C6.4.3.1

The following shall replace the first paragraph of AC6.4.3.1.

The ASTM standard for A 307 bolts covers three grades of fasteners, A, B and C. Grade A and B bolts may be used under these specifications as appropriate. Grade C bolts shall not be used as structural fasteners, but may be used for anchor bolts. There is no AASHTO standard corresponding to ASTM A 307.

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of 120 ksi for diameters 1/2 in. through 1 in. and 105 ksi for diameters 1 1/8 in. through 1 1/2 in., AASHTO M 164 (ASTM A 325)

- the Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength, AASHTO M 253 (ASTM A 490)

AASHTO M 253 (ASTM A 490) bolts are not allowed unless approved by the Chief Bridge Engineer.

Type 1 bolts should be used with steels other than weathering steel. Type 3 bolts conforming with either AASHTO M 164 (ASTM A 325) or AASHTO M 253 (ASTM A 490) shall be used with weathering steels. AASHTO M 164 (ASTM A 325), Type 1, bolts may be mechanically galvanized in accordance with AASHTO M 298 (ASTM B 695), Class 50, when approved by the Engineer. Galvanized bolts shall be retested after galvanizing, as required by AASHTO M 164 (ASTM A 325).

AASHTO M 253 (ASTM A 490) bolts shall not be galvanized.

Washers, nuts and bolts of any assembly shall be galvanized by the same process. The nuts should be overlapped to the minimum amount required for the fastener assembly, and shall be lubricated with a lubricant containing a visible dye.

Anchor bolts shall conform to ASTM F1554.

6.4.3.2 Nuts

6.4.3.2.1 Nuts Used with Structural Fasteners

The following shall replace A6.4.3.2.1.

Nuts used with structural fasteners shall conform to the following as appropriate.

Except as noted below, nuts for AASHTO M 164 (ASTM A 325) bolts shall conform to either the Standard Specification for Carbon and Alloy Steel Nuts, AASHTO M 291 (ASTM A 563), Grades DH, DH3, C, C3, and D, or the Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, AASHTO M 292 (ASTM A 194), Grades 2 and 2H.

Nuts for AASHTO M 253 (ASTM A 490) bolts shall conform to the requirements of AASHTO M 291 (ASTM A 563), Grades DH and DH3 or AASHTO M 292 (ASTM A 194), Grade 2H.

Nuts to be galvanized shall be heat treated Grade DH. The provisions of D6.4.3.1 shall apply. All galvanized nuts shall be lubricated with a lubricant containing a visible dye.

Plain nuts shall have a minimum hardness of 89 HRB.

Nuts to be used with AASHTO M 164 (ASTM A 325), Type 3 bolts shall be of Grade C3 or DH3. Nuts to be used with AASHTO M 253 (ASTM A 490), Type 3, bolts shall

COMMENTARY

The following shall supplement AC6.4.3.1.

A Lehigh University study shows that AASHTO M 253 (ASTM A 490) bolts are more sensitive to the number of threads in the grip than AASHTO M 164 (ASTM A 325) bolts. The decrease in tension in AASHTO M 253 (ASTM A 490) bolts after the maximum tension is reached is much more rapid than the unloading experienced in the AASHTO M 164 (ASTM A 325) bolt assembly. Also, the AASHTO M 253 (ASTM A 490) bolts have reduced ductility compared to the AASHTO M 164 (ASTM A 325) bolt having the same length of thread in the grip. Hot-dipped galvanized bolts are not permitted due to concerns associated with the quality of the threads.

The following shall replace the last paragraph of AC6.4.3.1.

There is no AASHTO standard corresponding to ASTM F1554.

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be of Grade DH3.

6.4.3.2.2 Nuts Used with Anchor Bolts

The following shall replace A6.4.3.2.2.

Nuts used with anchor bolts shall conform to the following as appropriate.

Nuts for ASTM F1554 anchor bolts shall conform to AASHTO M 291 (ASTM A563) for appropriate grade and size of anchor bolt.

Nuts to be galvanized shall be heat treated Grade DH or DH3. The provisions of D6.4.3.1 shall apply. All galvanized nuts should be lubricated with a lubricant containing a visible dye.

6.4.3.3 Washers

The following shall replace A6.4.3.3.

Washers shall conform to the Standard Specification for Hardened Steel Washers, AASHTO M 293 (ASTM F 436).

The provisions of D6.4.3.1 shall apply to galvanized washers.

6.4.3.4 Alternative Fasteners

The following shall replace the first portion of the first sentence of A6.4.3.4.

Other fasteners or fastener assemblies, not specified heretofore, may not be used unless approved by the Chief Bridge Engineer,

6.4.3.5 Load Indicator Devices

The following shall supplement A6.4.3.5.

For additional requirements concerning load indicator devices, see Publication 408.

6.4.7 Stainless Steel

The following shall be added to the last sentence of the last paragraph of A6.4.7.

“...and approved by the Chief Bridge Engineer.”

6.5 LIMIT STATES**6.5.2 Service Limit State**

The following shall supplement A6.5.2.

Bridge foundations shall be investigated for stability in accordance with D2.6.4.4.2, D3.7 and PP7.2.3 at the service I limit state. Pile resistances shall be determined in accordance with A6.12.1.2.1, A6.12.1.2.2 and A6.12.2.2.1 with resistance factors equal to 1.0.

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6.5.4 Strength Limit State**6.5.4.2 Resistance Factors**

The following shall replace the pile resistance factors in A6.5.4.2.

- for axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary..... $\phi_c = 0.35$
- for axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary..... $\phi_c = 0.45$
- for axial resistance of piles bearing on soluble bedrock $\phi_c = 0.25$
- for axial resistance of steel portion of concrete filled pipe piles in compression..... $\phi_c = 0.35$
- for combined axial and flexural resistance of undamaged piles:
 - axial resistance $\phi_c = 0.60$
 - flexural resistance..... $\phi_f = 0.85$

6.6 FATIGUE AND FRACTURE CONSIDERATIONS**6.6.1 Fatigue****6.6.1.2 Load-Induced Fatigue***6.6.1.2.1 Application*

Delete the second sentence of the first paragraph of A6.6.1.2.1

6.6.1.2.2 Design Criteria

The following shall replace A6.6.1.2.2.

For load-induced fatigue considerations, each detail shall satisfy:

$$\gamma (PTF) (\Delta f) \leq (\Delta F)_n \quad (6.6.1.2.2-1)$$

where:

γ = load factor specified in Table A3.4.1-1 for the fatigue load combination, use Fatigue I load combination for Interstate and NHS bridges

(Δf) = the force effect, live load stress range due to the

C6.5.4.2

The following shall supplement AC6.5.4.2.

The basis for the resistance factors for driven steel piles is described in DC6.15.2.

For bridge foundations evaluated in accordance with D3.7 at the strength limit state, the pile resistance factors in D6.5.4.2 shall apply, as appropriate.

C6.6.1.2.1

Delete the first paragraph of AC6.6.1.2.1.

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passage of the fatigue load as specified in A3.6.1.4 (ksi)

$(\Delta F)_n$ = the nominal fatigue resistance as specified in A6.6.1.2.5 and D3.6.1.4 (ksi)

PTF = Pennsylvania Traffic Factor as given in Table 6.6.1.2.2P-1

Table 6.6.1.2.2P-1 - Pennsylvania Traffic Factor for Cases I, II and III

Case	Type of Road	ADTT	PTF
I	National Highway System	---	1.2
II	Freeways, Expressways, Major Highways and Streets	> 500	1.2
III	Other Highways and Streets not included in Case I or II	---	1.0

6.6.1.2.3 Detail Categories

The following shall replace the second paragraph of A6.6.1.2.3.

Except as specified herein for fracture critical members, where the projected 100-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is less than or equal to that specified in Table 6.6.1.2.3-2 for the component or detail under consideration, that component or detail should be designed for finite life using the Fatigue II load combination specified in Table A3.4.1-1. Otherwise, the component or detail shall be designed for infinite life using the Fatigue I load combination. The single-lane Average Daily Truck Traffic $(ADTT)_{SL}$ shall be computed as specified in A3.6.1.4.2 and D3.6.1.4.2.

C6.6.1.2.3

The following shall replace the fifth paragraph of AC6.6.1.2.3.

Where the design stress range calculated using the Fatigue I load combination is less than $(\Delta F)_{TH}$, the detail will theoretically provide infinite life. Except for Categories E and E', for higher traffic volumes, the design will most often be governed by the infinite life check. Table 6.6.1.2.3-2 shows for each detail category the values of $(ADTT)_{SL}$ above which the infinite life check governs, assuming a 100-year design life and one stress range cycle per truck.

The following shall supplement Article AC6.6.1.2.3

PennDOT's design life is considered to be 100 years. In the overall development of the AASHTO LRFD Specification, the design life has been considered to be 75 years. The provisions of this section have been revised to reflect a 100 year design life.

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The following shall replace Table A6.6.1.2.3-2.

Table 6.6.1.2.3-2—100-yr $(ADTT)_{SL}$ Equivalent to Infinite Life

Detail Category	100-Year $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)
A	400
B	645
B'	775
C	965
C'	560
D	1405
E	2645
E'	4865

6.6.1.2.5 Fatigue Resistance

The following shall replace Equation A6.6.1.2.5-3.

$$N = (365) (100) n (ADTT)_{SL} \quad (6.6.1.2.5-3)$$

C6.6.1.2.5

The following shall replace fourth paragraph of AC6.6.1.2.5.

PennDOT's design life is considered to be 100 years. In the overall development of the LRFD Specification, the design life has been considered to be 75 years. This is the reason that the 75 in Equation A6.6.1.2.5-3 has been replaced with 100 in Equation D6.6.1.2.5-3.

6.6.1.2.6P Restricted Use Details

Details defined as Category D, E or E' in Section 3 (Welded Joints) of Table A6.6.1.2.3-1 are considered unacceptable for new designs. Such details shall be excluded from new designs, except when approved by the Chief Bridge Engineer.

Girder or floorbeam flanges inserted through a slot cut in the web of an intersecting member and then welded to one or both sides of the web to provide continuity are not acceptable. Moreover, such flanges butted flush against the web of the intersecting member and then welded to it are unacceptable.

Details involving the intersection of the flange of one girder with the web of another girder are unacceptable because a significant embedded crack-like interface may remain between members after the welding. Such a defect can quickly propagate, causing premature failure.

6.6.1.3 Distortion-Induced Fatigue

C6.6.1.3

The following shall supplement AC6.6.1.3.

The interaction of primary and secondary components of steel bridge structures often results in cracking at unexpected locations in relatively short periods of time. Such cracking was first observed in the webs of girder-type bridges at short gaps between transverse web attachments and the girder flanges. Investigations of this type of crack

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6.6.1.3.4P Distortion-Induced Fatigue: Unacceptable Details and Acceptable Alternative Details

Members and fasteners shall be detailed to reduce the effect of repeated variations or reversals of stress due to out-of-plane deformations or secondary forces. Examples of details which have proven to be unacceptable, based upon this criteria, are shown in Figure 6.6.1.3.4P-1. Acceptable alternatives to these unacceptable details are shown in Figure 6.6.1.3.4P-2. These details do not include all possible variations of distortion-sensitive details, but they are considered typical and will provide guidance.

Lateral gusset plates near transverse stiffeners or coped around transverse stiffeners shall be rigidly attached to the transverse stiffener (either bolted or welded). If this rigid attachment is not provided, the potential for localized out-of-plane distortion cracking of the web is created near the juncture of the web and transverse stiffener.

If lateral bracing is required, the preferred approach is to attach the gusset plate to the flange as shown in BD-620M and BC-754M. Welding of the gusset plate to the stiffener must be detailed to prevent intersecting welds.

COMMENTARY

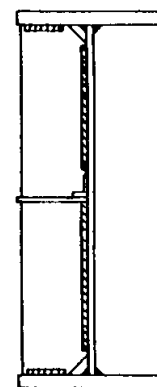
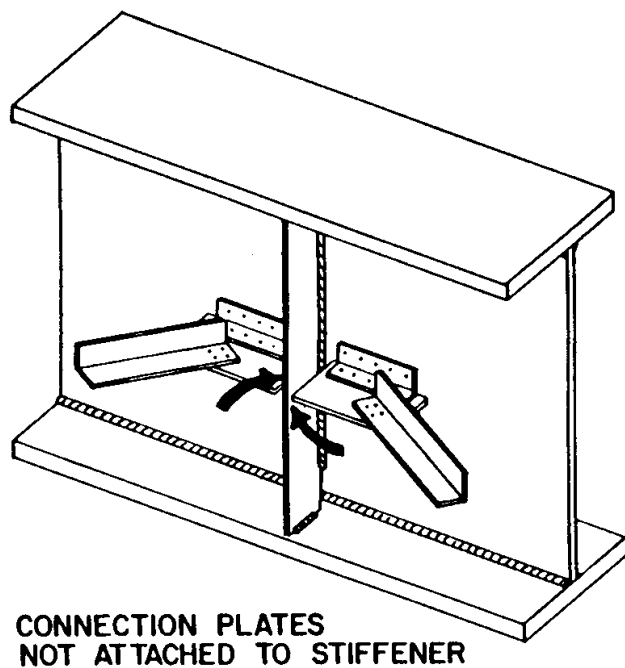
development suggest that the cracking is typical and is caused by out-of-plane displacements which result in large secondary web bending stresses. This is evident in the in-depth case studies presented by Mertz (1984).

Fatigue crack growth resulting from displacement-induced secondary stresses is difficult to anticipate, since it involves the actual behavior of a structure, rather than the assumed behavior. The differences between the actual and the assumed behavior are most critical at very localized regions, such as at the ends of cut-short transverse connection plates. The present design idealization does not account for such localized behavior.

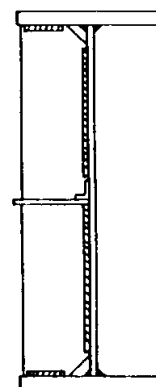
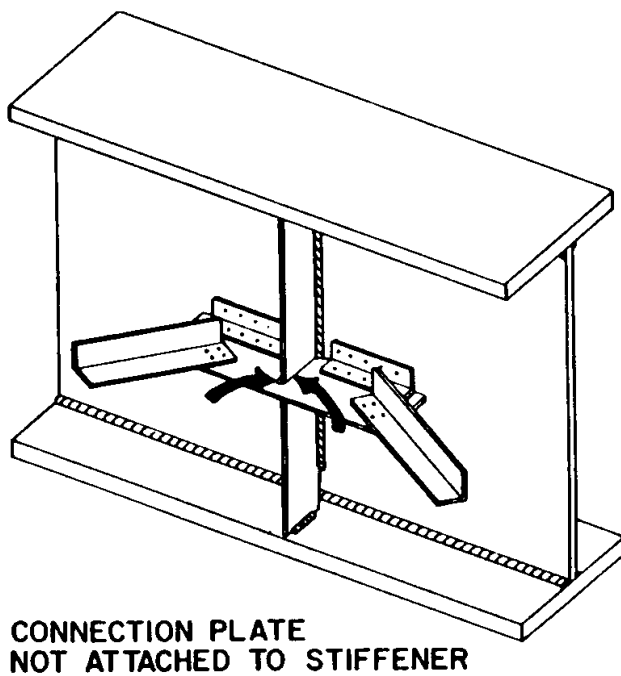
C6.6.1.3.4P

Rather than attempting to quantify the displacement-induced stresses and develop allowable values, it is the Department's philosophy that details susceptible to out-of-plane distortion are not acceptable. Through the design of better details, the inadequacy of the present design idealization in dealing with displacement-induced stresses is minimized.

Connection plates for either diaphragms or floorbeams shall be rigidly attached to both girder flanges (either bolted or welded). Cutting the connection plate short or merely providing a tight fit to the flange is not acceptable, since the potential for localized out-of-plane distortion cracking of the web exists near the juncture of the web and flange.

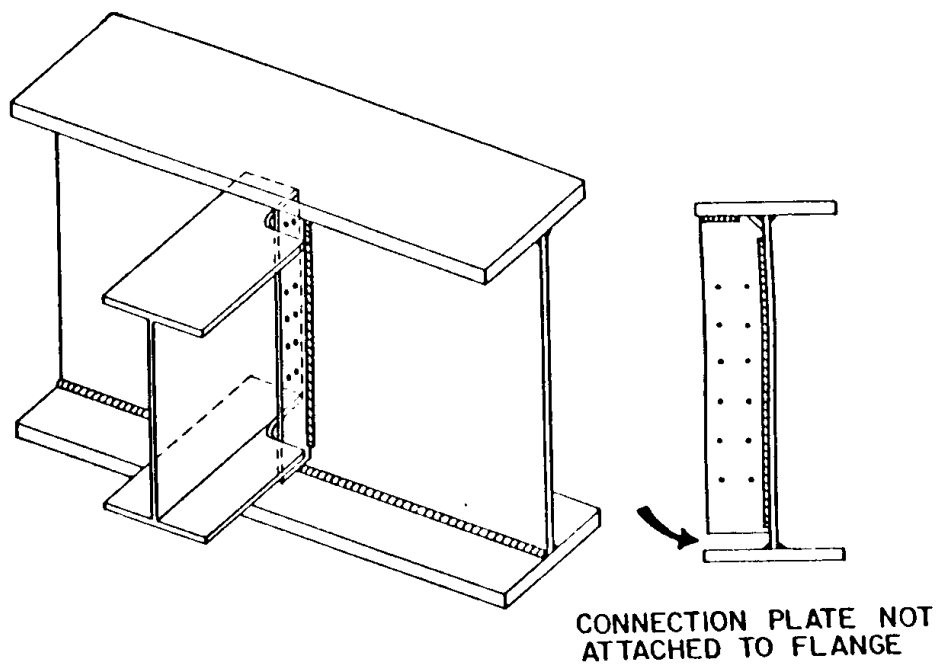


(a.) Lateral Connection Plate at Transverse Stiffener

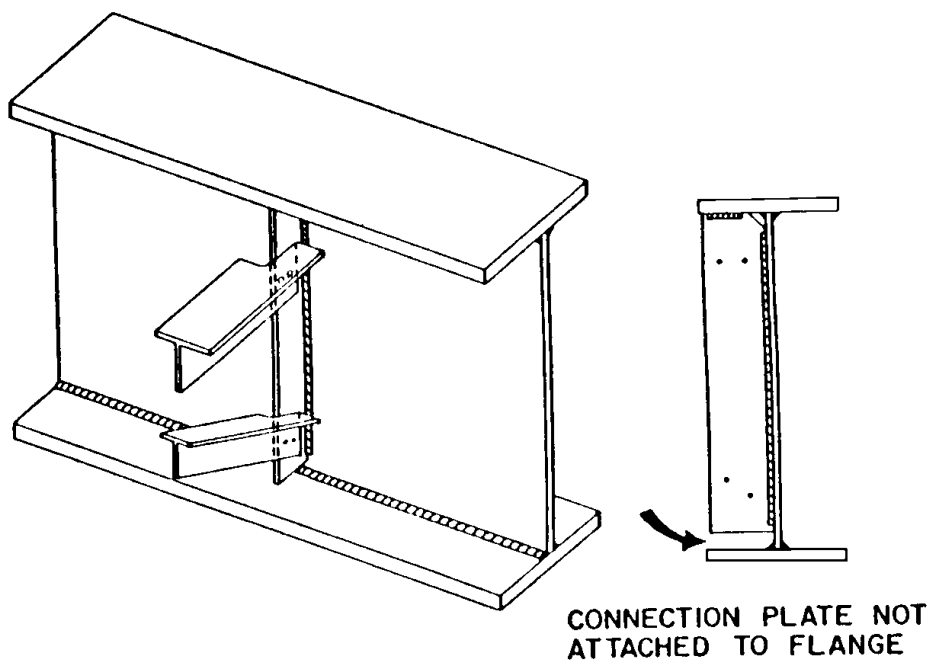


(b.) Lateral Connection Plate at Transverse Stiffener

Figure 6.6.1.3.4P-1 - Unacceptable Details

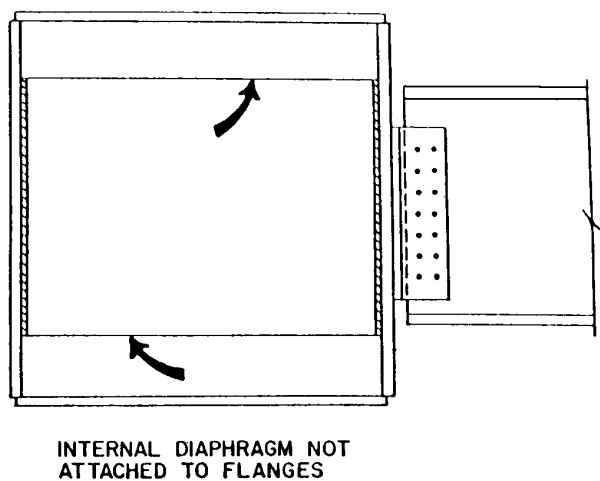


(c.) Girder- floorbeam Connection

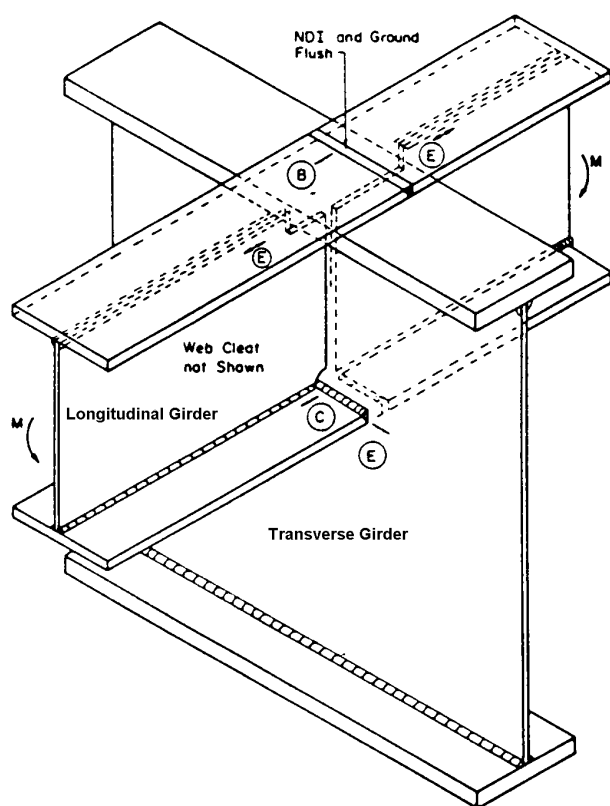


(d.) Girder-Cross - Frame Connection

Figure 6.6.1.3.4P-1 - Unacceptable Details (Continued)

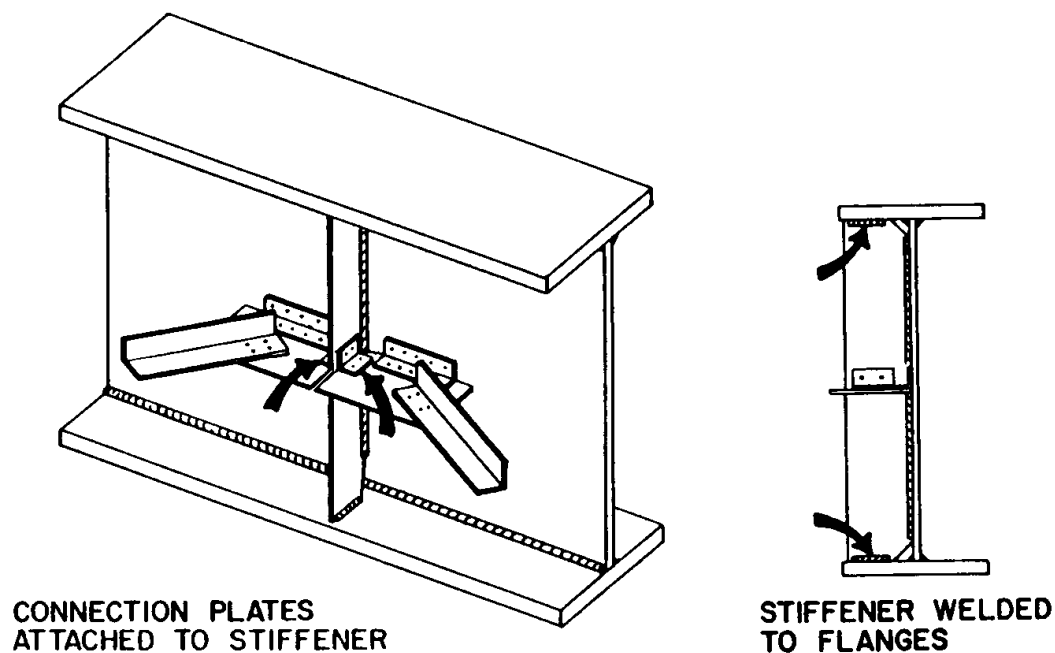


(e.) Internal Box Girder Diaphragm

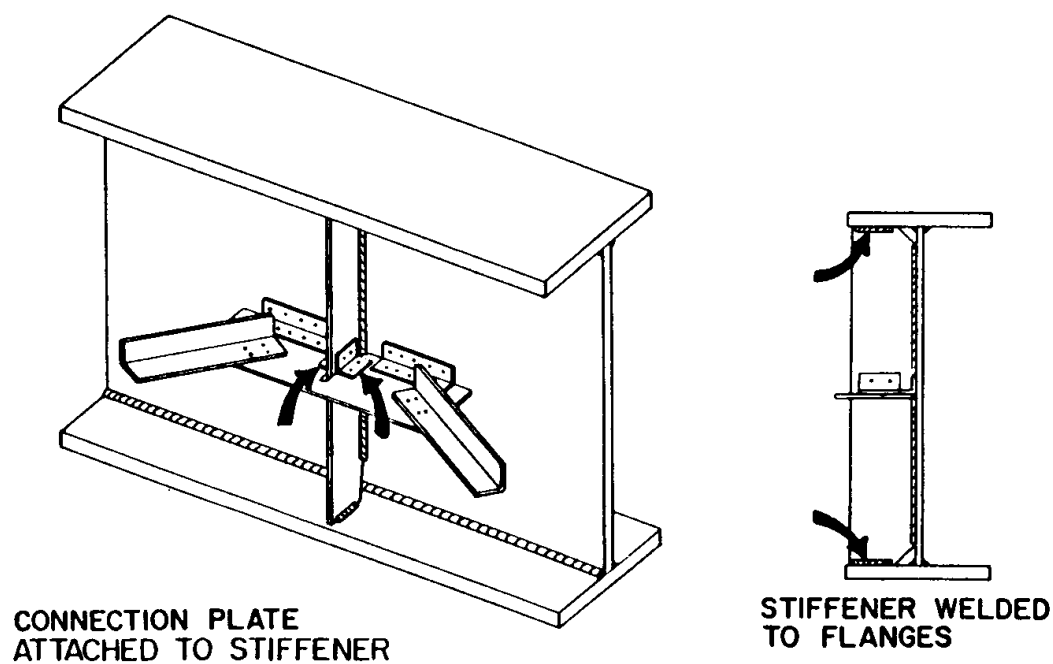


(f.) Longitudinal - to - Transverse Girder Connection

Figure 6.6.1.3.4P-1 - Unacceptable Details (Continued)

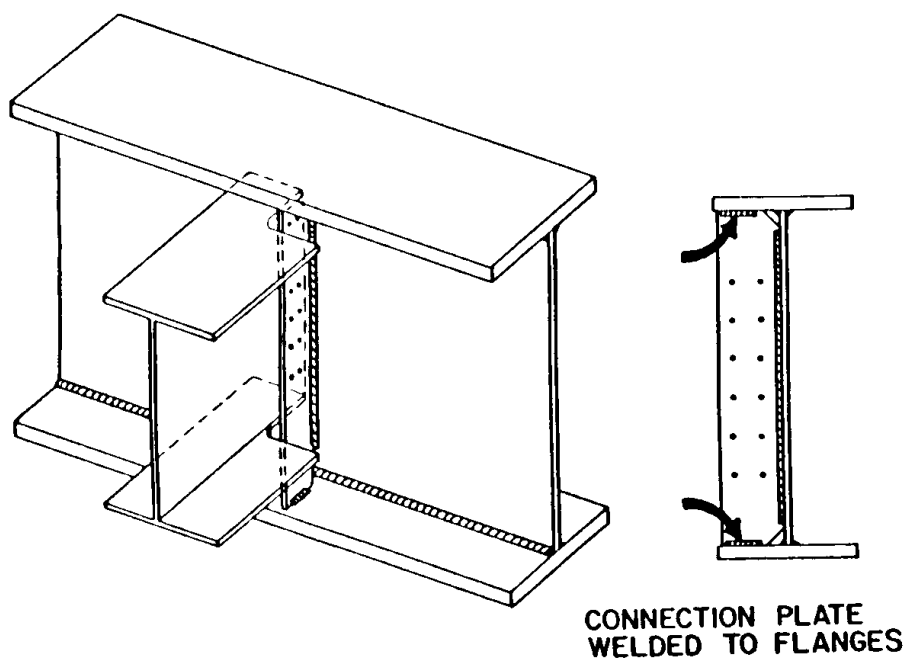


(a.) Lateral Connection Plate at Transverse Stiffener

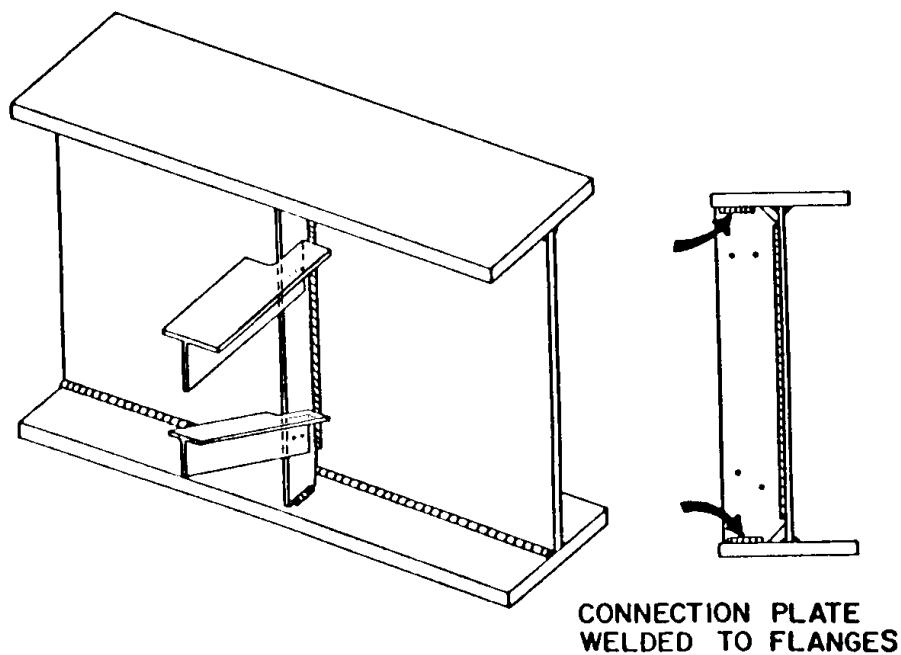


(b .) Lateral Connection Plate at Transverse Stiffener

Figure 6.6.1.3.4P-2 - Acceptable Alternative Details

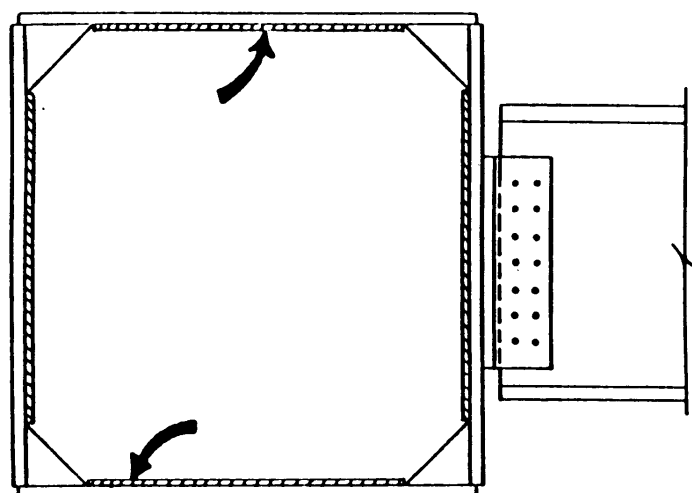


(c.) Girder Floorbeam Connection



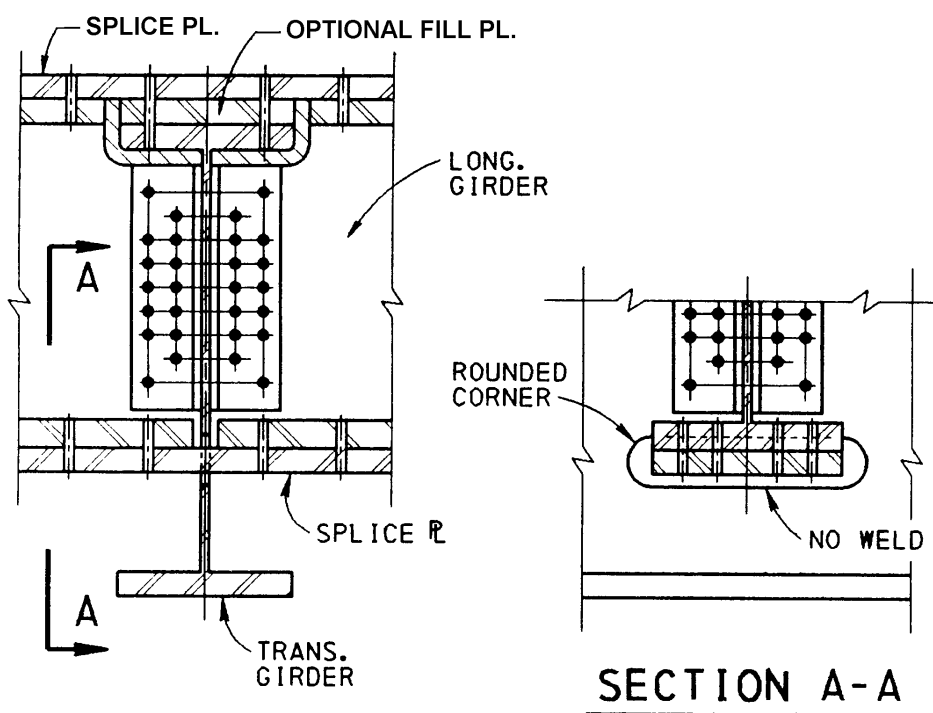
(d.) Girder - Cross-Frame Connection

Figure 6.6.1.3.4P-2 - Acceptable Alternative Details (Continued)



**INTERNAL DIAPHRAGM
WELDED TO FLANGES**

(e.) Internal box Girder Diaphragm



(f.) Slotted Component Through Which Another
Component Passes.

Figure 6.6.1.3.4P-2 - Acceptable Alternative Details (Continued)

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6.6.2 Fracture

The following shall supplement A6.6.2.

Charpy V-Notch tests shall be performed as specified as per Publication 408, Section 1105.02(a)5.

Diaphragms, cross-frames, bracing and connecting plates for curved girder bridges, straight girder bridges with skew less than 70°, or connections which are entirely welded and without any bolting are to be Charpy V-Notch tested.

Typical shop welded, field bolted diaphragms on straight bridges do not require Charpy V-Notch testing (unless bridge skew is less than 70°).

6.7 GENERAL DIMENSION AND DETAIL REQUIREMENTS**6.7.2 Dead Load Camber**

Delete the last paragraph of A6.7.2.

The following shall supplement A6.7.2.

Camber is provided for the beams so that after all the dead loads (not including the future wearing surface) are applied, the beam is at the proper elevation. Camber is not used for the control of live load deflections.

The designer shall add the following note, as appropriate for the structure type, in the General Notes section of the plans:

- For all straight I-girder bridges, including skewed bridges:
“Girder webs shall be plumb under the full dead load existing at the end of construction”
- For horizontally curved I-girder bridges:
“Girder webs shall be plumb under the dead load of the girders before any other loads are applied”

6.7.2.1P Camber Due to Weight of Deck Slab

The camber, due to the weight of deck slab, shall be determined from an analysis in which the weight of the deck slab is applied all at once.

6.7.2.2P Camber Details for Design Drawings

A diagram and a table of camber ordinates (see Figure 6.7.2.2P-1) shall be shown on the contract plans. Ordinates shall be provided for all beams at one-tenth points and at field splice points (at dead load points of contraflexure if field splices are not provided) to account for the following:

- (a) Weight of steel
- (b) Weight of deck slab (see D6.7.2.1P)

C6.7.2

Delete the third and fourth paragraphs of AC6.7.2 including the bullet list following each of the two paragraphs.

Full dead load includes the weight of pavement or overlays included in the initial construction. It does not include the future wearing surface.

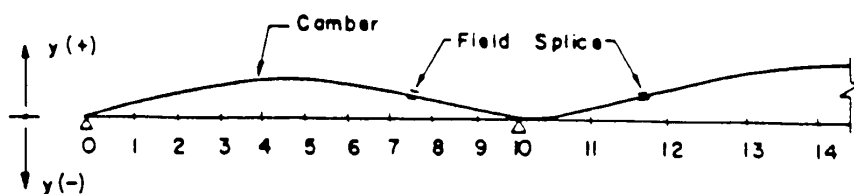
C6.7.2.2P

Rolled beams should not be specified when the total camber exceeds 8 in. due to fabrication issues to achieve this camber. For this case, detail a plate girder with similar properties.

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- (c) Superimposed dead load (do not include future wearing surface)
- (d) Vertical curve
- (e) Superelevation
- (f) 50% of heat curve camber (see D6.7.2.3P)
- (g) Total due to above



CAMBER ORDINATES y, INCHES

		POINT NO.							
Beam No.		1	2	3				Field Splice	
* Due to wt. of Steel	1								
	2								
	3								
* Due to wt. of Deck	1								
	2								
	3								
* Due to Super Load	1								
	2								
	3								
* Due to Curve	1								
	2								
	3								
Total	1								
	2								
	3								

* Dead load deflection = - (Values tabulated)

Figure 6.7.2.2P-1 - Camber Details

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When total camber is less than the minimum that can be maintained in a beam (WF sections), no camber is required, but the following note shall be shown on the contract plans:

“Beams shall be placed with any mill camber up; the contractor shall consider and compensate for dead load deflection, due to the weight of the concrete, when forming and constructing the deck slab.”

Designers shall show theoretical dead load deflection data on plans even when no special camber is to be fabricated into the beams (i.e., when using mill camber), since this information is required by the contractor to construct the deck to the correct finished deck elevation.

The requirements for cross-section elevations at 10 ft. intervals along the length of girder bridges are found in PP1.6.4.11(e).

6.7.2.3P Heat-Curve Camber Corrections

To compensate for possible loss of camber of heat-curved girders in service as residual stresses dissipate, the amount of camber in inches, Δ , at any section along the length of the girder shall be equal to:

$$\Delta = \frac{\Delta_{DL}}{\Delta_M} (\Delta_M + \Delta_R) \quad (6.7.2.3P-1)$$

where:

For $R < 1,000$ ft.

$$\Delta_R = \frac{0.02 L^2 F_y}{E Y_o} \left(\frac{1,000 - R}{850} \right) \quad (6.7.2.3P-2)$$

For $R > 1,000$ ft.

$$\Delta_R = 0$$

R = radius of curvature of curved girder (ft.)

E = modulus of elasticity of the girder flange (ksi)

F_y = specified minimum yield strength of the girder flange (ksi)

Y_o = distance from the neutral axis to the extreme outer fiber (in.)

L = span length for simple span; the distance between a simple end support and the dead load contraflexure point or the distance between points of dead load contraflexure points for continuous spans (in.)

C6.7.2.3P

Part of the heat-curve camber loss is attributable to construction loads and will occur during construction of the bridge; total heat-curve camber loss will be complete after several months of in-service loads. Therefore, a portion of the heat-curve camber increase (approximately 50%) should be included in the bridge profile. Camber losses of this nature (but generally smaller in magnitude) are also known to occur in straight beams and girders.

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Δ_{DL} = camber at any point along L (in.)

Δ_M = maximum value of Δ_{DL} within L (in.)

Heat-curve camber loss between dead load contraflexure points adjacent to piers is small and may be neglected.

6.7.3 Minimum Thickness of Steel**C6.7.3P**

The following shall replace the first two paragraphs of A6.7.3.

Structural steel (including bracing, cross-frames, and all types of gusset plates), except for webs of certain rolled shapes, closed ribs in orthotropic decks, fillers and in railings shall not be less than 3/8 in. in thickness. For girders, the minimum flange plate thickness shall be 3/4 in. unless the fabricator can demonstrate the ability to satisfactorily fabricate and erect plate girders with thinner flange plates. For girders with longitudinal stiffeners, the minimum web thickness shall be 1/2 in.. The web thickness of rolled beams or channels shall not be less than 0.23 in.. The thickness of closed ribs in orthotropic decks shall not be less than 3/16 in.

For girder flanges, bearing stiffeners and splice plates for bridges that are to be metallized, the width of the plates are to be oversized by 1/8 in. to account for edge grinding. The flange, bearing stiffener plates and splice plates to be shown on the plans shall be the oversized plates. For metallized bridges, the estimated quantity of fabricated structural steel shall be based on the oversized plates.

This requirement of minimum web thickness for girders with longitudinal stiffeners was added to avoid web buckling and oil canning of deep girders. PennDOT has previously used 3/8 in. thickness resulting in web oil canning effect, specifically on I-476 over Conestoga Avenue. The New York Department of Transportation has successfully used the specified criteria.

For metallized bridges, the rolled edges of angles, channels and wide flange beams do not require edge grinding, therefore these components are not to be oversized.

6.7.4 Diaphragms and Cross-Frames**6.7.4.1 General****C6.7.4.1**

The following shall supplement the first paragraph of A6.7.4.1.

The maximum spacing of cross-frames or diaphragms shall be 25 ft..

The following shall supplement AC6.7.4.1.

The following shall supplement A6.7.4.1.

Skew effects must be considered when designing diaphragms, especially when the skew angle is less than 70°. Proper consideration of unbraced length and diaphragm loads from non-uniform deflections is mandatory. Design calculations must consider the fact that cross-frames in skewed bridges connect different points of the span of adjacent girders and that these points will not deflect the same amount. Therefore, a check considering these differences must be made, and the resulting design forces must be used in the cross-frame design.

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For sharply skewed bridges (typically, skews less than or equal to 60°), a cross-frame or diaphragm normal to the girder shall be located such as to minimize the effects of differential deflections, while satisfying the minimum cross-frame or diaphragm spacing requirement.

For additional analysis criteria for bridges with skew angles less than 70°, see D4.6.2.2.1.

Solid plate diaphragm members in horizontally curved and skewed bridges may be used at support locations.

6.7.4.2 I-Section Members

The following shall supplement the first paragraph of A6.7.4.2.

Cross frames and steel girders in curved and straight skewed bridges should be detailed so that the webs are plumb under a specified loading condition.

For plate girder depths 40 in. and less, the diaphragm depth may be reduced to 0.5 of the girder depth.

The following shall replace the fourth and fifth paragraphs of A6.7.4.2.

Where supports are skewed between 90° and 70°, intermediate diaphragms or cross-frames may be placed in contiguous skewed lines parallel to the skewed supports.

Where supports are skewed less than 70°, diaphragms or cross-frames shall be normal to the girders and may be placed in contiguous or discontinuous lines. The Department prefers contiguous lines, except near skewed interior supports. At skewed interior supports, eliminate diaphragms or cross-frames where possible; however, adequate diaphragms or cross-frames must be present to brace the girders. When staggered (discontinuous) lines of diaphragms or cross-frames are used, the girder capacity must be reduced to account for the lateral flange bending.

If the supports are skewed, end cross-frames need not be co-linear with the line of bearings, see Standard Drawing BC-754M. For additional skewed cross-frame requirements, see D6.7.4.1.

COMMENTARY

In locating intermediate cross-frames or diaphragms in sharply skewed bridges, the designer must consider distinct issues associated with each girder connected by the cross-frame or diaphragm. A cross-frame or diaphragm close to the bearing on one girder line may introduce forces into the system (cross-frame or diaphragm and girder flange) due to “nuisance stiffness,” where the deflection of one girder line cannot match the adjacent girder line. In these cases, elimination of a cross-frame or diaphragm is advisable. In addition, the initial cross-frame or diaphragm must be located such that the maximum permitted spacing is not exceeded in the adjacent connected girder. In some cases, the first interior line of cross-frames or diaphragms may not be full width across the superstructure, and the number of bays along a girder length may not be constant for each girder in the superstructure.

Solid plate diaphragms can be used at support locations in horizontally curved and skewed bridges. Replacing cross frames with solid plate diaphragms at abutment and pier locations has been shown to not adversely affect or appreciably benefit deformations during construction. For other intermediate locations along the bridge spans, solid plate diaphragms may cause higher stresses and deformations in the bridge structures during construction when compared to the use of cross frames.

C6.7.4.2

The following shall supplement AC6.7.4.2.

End cross-frames must be parallel to centerline of bearings, but need not coincide with bearing line.

In curved and straight skewed bridges, cross-frame stresses can increase appreciably as a result of the locked-in stresses caused by inconsistent detailing. This increase may be more pronounced in bridges with larger cross frame spacings and also for cross frames near the bridge supports. For straight skewed bridges, develop shop drawings which detail all webs plumb after the full dead load (self weight of all structural and non-structural components, not including weight of the future wearing surface) is applied. For curved bridges, develop shop drawings and erection procedures which detail all webs plumb when girders are erected and diaphragms connected. See standard drawing BC-754M.

Placement of cross frames parallel to the skew has been shown to induce significant localized lateral bending near support locations.

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COMMENTARY

The following shall replace the last paragraph of A6.7.4.2.

In no case shall L_b exceed 25 ft.

6.7.4.3 Box Section Members

The following shall replace the fourth paragraph of A6.7.4.3.

Intermediate internal diaphragms or cross-frames shall be provided. For all single box sections, horizontally curved sections, and multiple box sections in cross-sections of bridges not satisfying the requirements of A6.11.2.3 or with box flanges that are not fully effective according to the provisions of A6.11.1.1, the internal bracing shall be spaced to control cross-section distortion, with the spacing not to exceed 25 ft.

6.7.5 Lateral Bracing**6.7.5.2 I-Section Members****6.7.7 Heat-Curved Rolled Beams and Welded Plate Girders****6.7.7.2 Minimum Radius of Curvature**

The following shall replace A6.7.7.2.

For heat-curved beams and girders, the horizontal radius of curvature measured to the centerline of the girder web shall not be less than 150 ft and shall not be less than the larger of the values determined from the following two equations:

C6.7.4.3

The following shall replace the first sentence of the fourth paragraph of AC6.7.4.3.

Cross-sectional distortion stresses are typically controlled by the internal cross-frames or diaphragms, with the spacing of these members not to exceed 25.0 ft.

The following shall replace the first sentence of the seventh paragraph of AC6.7.4.3.

In tub sections with inclined webs with a slope exceeding 1 to 4, additional intermediate internal cross-frames, diaphragms, or struts may be required to increase the resistance of discretely braced top flanges of tub sections to lateral bending resulting from a uniformly distributed transverse load acting on the flanges.

C6.7.5.2

The following shall supplement AC6.7.5.2.

Wherever possible, horizontal lateral bracing in or near the plane of the bottom flange should be eliminated from bridges, and the girders should be designed to carry the wind load between diaphragms according to A4.6.2.7.1. Horizontal lateral bracing is relatively expensive because of the detail associated with it. Furthermore, there are often forces associated with horizontal lateral bracing which can result in distortion-induced fatigue; these forces are also a significant factor on steel bridges. Therefore, horizontal lateral bracing should not be considered for the improvement of redundancy.

When horizontal lateral bracing is required, it should be attached to the bottom flange wherever practical. (BD-620M permits attachment to the top flange.)

C6.7.7.2P

The stress analysis performed by Brockenbrough (1970) to develop the equations in earlier editions of these specifications was based on the following assumptions:

- The heat curving process introduces heat continuously along the girder length, resulting in a heated portion of the flange which is the same at every cross section along the length.

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$$R = 0.0365 \frac{b}{\psi} \left(\frac{D_w}{t_w} \right)^2 \quad (6.7.7.2-1)$$

$$R = \frac{12,800 b}{F_{yw} \psi} \quad (6.7.7.2-2)$$

Additionally, for singly-symmetric heat-curved beams and girders, the horizontal radius of curvature measured to the centerline of the girder web shall not be less than that determined from:

$$R = \left[-0.413(\phi v)^3 + 0.930(\phi v)^2 - 0.710 \phi v + 0.305 - \frac{\psi}{30} \right] \psi^2 \left(\frac{E}{F_{yw}} b \right) \quad (6.7.7.2-3P)$$

in which:

$$v = \frac{t_{tf}}{t_f} \quad (6.7.7.2-4P)$$

$$\phi = \frac{b_{tf}}{b} \quad (6.7.7.2-5P)$$

and

$$\psi = \frac{b_{tf} t_{tf} + b t_f + D_w t_w}{b_{tf} t_{tf} + b t_f} \quad (6.7.7.2-6P)$$

where:

b = width of wider flange (in.)

b_{tf} = width of narrower flange (in.)

D_w = clear distance between flanges (in.)

F_{yw} = specified minimum yield strength of a web (ksi)

R = radius of curvature (in.)

t_w = web thickness (in.)

t_f = thickness of wider flange (in.)

t_{tf} = thickness of narrower flange (in.)

The proportions of a heat curved singly-symmetric girder shall be such that the value of v given by Eq. 6.7.7.2-4P and ϕ given by Eq. 6.7.7.2-5P shall each not be less than 0.4 nor greater than 1.0.

In addition to the above requirements, the radius shall not be less than 1,000 ft when the flange thickness exceeds 3.0 in. or the flange width exceeds 30.0 in.

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- The girder cross-section is a doubly-symmetric I-shaped section.
- A tensile shrinkage force P develops in the heated portion of each flange near the flange edge on the inside of the curve. P is eccentric to the girder centroid.
- The heated width of the flange is $0.20b$, i.e. 20 percent of the flange width.
- The entire cross-section of the girder resists P , resulting in transverse bending stresses associated with the horizontal curvature about a vertical axis and axial compressive stresses on the cross section. The entire cross-section remains elastic and plane sections remain plane.

The resulting equation for the compressive stress in the web due to heat curving, normalized to the yield stress, was

$$\frac{\sigma_w}{F_{yw}} = \frac{-6,000}{F_{yw}} \frac{1}{\psi} \frac{1}{R/b} \quad (C6.7.7.2P-1)$$

The new equations in this Article are based on studies reported in Modjeski and Masters and Lehigh University (2011) based on numerical simulations that overcame three limitations of the stress analysis by Brockenbrough. These three limitations are:

- The stress in the cross-section was permitted to exceed the yield stress,
- The stress analysis was limited to doubly-symmetric cross-sections, and
- The heated width was fixed at $0.20b$, rather than varying with R .

The new studies which varied the heated flange width with R and considered the effect of yielding in the flanges, found that the compressive stress in the web due to heat curving could be adequately represented by the empirical equation of Eq. C6.7.7.2P-1 with the constant 6,000 replaced by 6,670 resulting in Eq. C6.7.7.2P-2.

$$\frac{\sigma_w}{F_{yw}} = \frac{-6,670}{F_{yw}} \frac{1}{\psi} \frac{1}{R/b} \quad (C6.7.7.2P-2)$$

Singly-symmetric cross-sections were included with the limitation that the moment of the heated area of the narrower flange about the elastic neutral axis is equal to the moment of the heated area of the wider flange about the elastic neutral axis. Parametric studies demonstrated that Eq. C6.7.7.2P-1 with the constant equal to 6,670 was

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adequate for the singly-symmetric case as well, provided that the flange width in the equation is taken as the width of the wider flange.

With the stress in the web from the heat curving operation now quantified, limits on R were established following the process developed by Brockenbrough. The basis for Eq. D6.7.7.2-1 is limiting stress in web to the buckling stress instead of the post buckling strength previously used. Eq. D6.7.7.2-2 is based on Von Mises yield criteria but revised assuming greater web shear stresses ($0.425 F_y$) under current design practice and an allowable stress of $0.90 F_y$, while the original development assumed a web shear stress of $0.33 F_y$ and an allowable stress of F_y .

The stress analysis of singly-symmetric heat-curved girders that was used to develop Eq. D6.7.7.2-1 and Eq. D6.7.7.2-2 is valid when R is greater than the radius at which the heated width of the narrower flange equals the flange half width. This limit, provided by Eq. 6.7.7.2-3P, is also considered to be a practical limit on heat curving. For many cases, this limit on heated flange width is reached when the web stresses are quite high, so it is not of practical concern, as the radius will be limited by the first two equations. However, for highly unsymmetrical cases, this limit will be reached when the web stresses are not large, so that a limit on the radius of heat curved girders based on this limit on heated flange width is needed.

The specified limits on v and ϕ define the range of variables for which the analyses leading to Eqs. D6.7.7.2-1, D6.7.7.2-2 and 6.7.7.2-3P were run.

Guidance on shop implementation of continuous and V-heating to heat curve girders is given in Brockenbrough (1972) and (1973).

6.8 TENSION MEMBERS**6.8.2 Tensile Resistance****6.8.2.2 Reduction Factor, U**

The following shall replace the second paragraph of A6.8.2.2.

The reduction factors, specified in A6.8.2.2, shall be used to account for shear lag. Reduction factors developed from refined analysis or tests may be used if approved by the Chief Bridge Engineer.

6.8.3 Net Area**C6.8.3**

Delete the first paragraph of AC6.8.3.

6.9 COMPRESSION MEMBERS**6.9.5 Composite Members**

SPECIFICATIONS

COMMENTARY

6.9.5.1 Nominal Compressive Resistance

The following shall replace the definition of n in A6.9.5.1.

n = modular ratio of the concrete as specified in D5.4.2.1

6.10 I-SECTION FLEXURAL MEMBERS**6.10.1 General**

The following shall supplement A6.10.1.

Open-framed systems are those which have no horizontal lateral bracing in or near the plane of the bottom flange. Lateral bracing, when used, is provided to resist wind loads, but it is generally not needed since the girders can be designed to carry wind loads between the diaphragms.

If horizontal lateral bracing is included, the open-framed system distribution factors shall be used. If a horizontal lateral bracing system is used, the connections must be detailed to ensure that the fatigue life of the bracing system is at least that of the girder.

Although the lateral wind bracing may not be required for the final constructed condition, the need for lateral wind bracing during construction shall be investigated.

If girders deeper than 14 ft. are necessary for economical design or esthetic considerations, they shall be designed with a horizontal field splice. The approval of the

C6.10.1

Delete the reference to Appendix B6 from the sentences in the second and third paragraphs of AC6.10.1.

The following shall replace the second and third sentences in the third paragraph of AC6.10.1.

For the majority of straight non-skewed bridges, flange lateral bending effects tend to be most significant during construction and tend to be insignificant in the final constructed condition. Significant flange lateral bending may be caused by wind, by torsion from eccentric concrete deck overhang loads acting on cantilever forming brackets placed along exterior girders, and by the use of staggered cross-frames in conjunction with skews less than 70°.

The following shall supplement AC6.10.1

The application of open-framed system distribution factors for closed-framed systems is generally conservative.

If horizontal lateral bracing system is used, a rational analysis may consider a reduction in lateral live load distribution factor due to the quasi-box action of the closed-frame system.

The design procedure for evaluating the need for lateral bracing during construction shall be per BD-620M. As agreed upon by the APC Subcommittee for Steel Bridge Superstructures, the contractor is responsible for stability of the girders during erection, including providing wind bracing during erection as needed. This responsibility includes the analysis, design, material, fabrication and installation (and removal) of wind bracing during erection at no cost to the Department.

Any reduction in live load distribution factor must be approved by the Chief Bridge Engineer.

Flanged web splices (i.e., those constructed with angles) are not desirable.

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Chief Bridge Engineer shall be required when a horizontal field splice is incorporated. The contract plans shall include a statement to permit the elimination of the horizontal field splice at the Contractor's option.

6.10.1.1 Composite Sections*6.10.1.1.1 Stresses*

The following shall supplement A6.10.1.1.1

If concrete with expansive characteristics is used, composite design shall be used with caution, and provision must be made in the design to accommodate the expansion.

Composite section properties (see A6.10.1.1.1b, D6.10.1.1.1b and D6.10.1.1.1c) shall be assumed in the positive and negative moment regions for the calculation of design moments, shears and deflections.

6.10.1.1.1a Sequence of Loading

The following shall replace last paragraph of A6.10.1.1.1a

For unshored construction, permanent load applied before the concrete deck has attained 75% of its compressive strength shall be assumed carried by the steel section alone; permanent load and live load applied after this stage shall be assumed carried by the composite section. For shored construction, all permanent loads shall be assumed applied after the concrete deck has hardened or has been made composite and the contract documents shall so indicate.

Use of shored systems requires the prior approval of the Chief Bridge Engineer.

For continuous spans, the final dead load moment at each design section shall be taken as the greater of either the dead load moment considering the weight of the concrete deck to be instantaneously applied or a moment based upon an incremental analysis of the specified slab placement sequence. Similarly, stresses should be computed based on the more critical of the incremental and instantaneously applied loads.

6.10.1.1.1b Stresses for Sections in Positive Flexure

Delete Equation A6.10.1.1.1b-1

The following shall supplement A6.10.1.1.1b:

For normal and low density concrete, the modular ratio is given in D5.4.2.1.

6.10.1.1.1c Stresses for Sections in Negative Flexure

The following shall replace A6.10.1.1.1c

For calculating flexural stresses in sections subjected to negative flexure, the composite section for both short-term

C6.10.1.1.1P

If the concrete is expansive, estimate expansion and properly design concrete to flange connection by adding additional shear studs.

C6.10.1.1.1a

Delete the first paragraph of AC6.10.1.1.1

C6.10.1.1.1b

The following shall replace AC6.10.1.1.1b

It is preferable to proportion composite sections in simple spans and the positive moment regions of continuous spans so that the neutral axis lies below the top surface of the steel beam.

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and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck.

Cut-off points for the main reinforcement in cast-in-place decks over interior supports for continuity shall be staggered as required by design.

Concrete on the tension side of the neutral axis shall not be considered in calculating resisting moments.

6.10.1.1.1d Concrete Deck Stresses

The following shall replace A6.10.1.1.1d

For calculating longitudinal flexural stresses in the concrete deck in the positive moment region due to transient loads, the short-term modular ratio, n , shall be used. For calculating longitudinal flexural stresses in the concrete deck in the positive moment region due to permanent loads, the long-term modular ratio, $3n$, shall be used.

The deck concrete shall not be considered effective in resisting negative moments.

6.10.1.1.1fP Lateral Support of Top Flanges Supporting Timber Decks

The compression flanges of girders supporting timber floors shall not be considered to be laterally supported by the flooring, unless the floor and fastenings are specially designed to provide such support. Laminated timber decks shall be provided with steel clips designed to furnish adequate lateral support to the top flange.

6.10.1.2 Noncomposite Sections

The following shall supplement A6.10.1.2

Whenever technically feasible, all structures shall be made composite.

6.10.1.3 Hybrid Sections

The following shall supplement A6.10.1.3.

The use of girders with web yield strength higher than the flange yield strength requires the prior approval of the Chief Bridge Engineer.

6.10.1.4 Variable Web Depth Members

The following shall supplement A6.10.1.4.

The use of girders with variable web depth requires the prior approval of the Chief Bridge Engineer.

C6.10.1.1.1d

The following shall replace AC6.10.1.1.1d.

The requirements of D6.10.1.1.1c are such that the deck concrete is not considered effective in resisting tension under any load combination.

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6.10.1.5 Stiffness

The following shall supplement A6.10.1.5.

In the computation of flexural stiffness and flexural resistance of beams, the haunch shall be taken as zero. However, in the computation of dead load, the haunch shall be taken into account.

6.10.1.7 Minimum Negative Flexure Concrete Deck Reinforcement

The following shall replace A6.10.1.7

In negative flexure regions of any continuous span, the total cross-sectional area of the longitudinal reinforcement shall not be less than 1 percent of the total cross-sectional area of the slab. The reinforcement used to satisfy this requirement shall have a specified minimum yield strength not less than 60 ksi and a size not exceeding No. 6 bars.

The required reinforcement shall be placed in two layers uniformly distributed across the slab width, and two-thirds shall be placed in the top layer. The individual bars shall be spaced at intervals not exceeding 12 in within each row.

Shear connectors shall be provided along the entire length of the girder for new bridges and rehabilitated bridges (i.e. where feasible on deck replacement projects) to develop stresses in the plane joining the concrete and steel in accordance with A6.10.10 and D6.10.10.

6.10.1.9 Web Bend-Buckling Resistance*6.10.1.9.3P Nominal Flexural Resistance*

The compressive bending stress due to unfactored dead load and construction loads in webs with or without longitudinal stiffeners shall be limited as follows:

$$f_{cw} \leq \frac{5.4 E}{\gamma_w \left(\frac{D_c}{t_w} \right)^2} \quad (6.10.1.9.3P-1)$$

where:

COMMENTARY

C6.10.1.5

The following shall replace the second paragraph of AC6.10.1.5.

Field tests of composite continuous bridges have shown that there is considerable composite action in negative bending regions (Baldwin et al. 1978; Roeder and Eltvik 1985; Yen et al. 1995). Therefore, the stiffness of the full composite section is to be used over the entire bridge length for the analysis of composite flexural members, but not for stress calculations.

The following shall supplement AC6.10.1.5.

Field measured haunch depths may be used in the computation for flexural stiffness and resistance when rating existing bridges.

Other stiffness approximations which are based on sound engineering principles may be used if approved by the Chief Bridge Engineer.

C6.10.1.7

Delete the second paragraph and the last paragraph of AC6.10.1.7

The following shall replace the last sentence in the third paragraph of AC6.10.1.7.

The above applies for members that are designed by the provisions of Article A6.10, Article D6.10 or Appendix A6.

C6.10.1.9.3P

Equation 6.10.1.9.3P-1 is derived from the basic plate buckling formula for Case 5, shown in Figure C6.10.1.9.3P-1. Case 5 addresses buckling due to flexural compressive stress in a plate girder. Note that transverse stiffener spacing has little influence on flexural buckling.

For $\gamma_w = 1.0$, f_{cw} is the critical buckling stress. Since all webs have some initial out-of-flatness, buckling occurs at a stress smaller than the critical buckling stress. Buckling does not occur suddenly; rather, lateral deflection of the web increases as the moment increases. After buckling occurs,

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f_{cw} = compressive bending stress in the web due to unfactored dead load and construction loads (ksi)

The calculation of f_{cw} shall consider (but is not limited to) the following:

- steel girder weight
- concrete deck weight
- deck form weight
- construction equipment weight
- concrete haunch weight
- pouring sequence effect
- camber effect

E = modulus of elasticity of steel (ksi)

D_c = For webs without longitudinal stiffeners, clear distance between the neutral axis of the non-composite section for dead load and the compression flange. For webs with longitudinal stiffeners, clear distance between the neutral axis of the section for dead loads and the longitudinal stiffener (in.)

t_w = web thickness (in.)

γ_w = 1.0 where diaphragms or cross-frames are not staggered; 1.3 where diaphragms or cross-frames are staggered; as required by the Chief Bridge Engineer for other conditions

For sections where the slenderness limit given by Equation 6.10.1.9.3P-2 are not met, the factored shear resistance, V_r , shall be greater than or equal to four times the shear due to the unfactored dead load. For non-composite sections not meeting the requirements of Equation 6.10.1.9.3P-2, V_r shall be greater than or equal to four times the total unfactored shear, including live load.

$$\frac{5.4 E}{f_{cw} \left(\frac{D_c}{t_w} \right)^2} \geq 2.50 \quad (6.10.1.9.3P-2)$$

COMMENTARY

the flexural stress carried by the web is redistributed to the compression flange.

The LRFD Specification may allow flexural buckling to occur to various degrees under construction loading before composite action develops. This buckling, together with initial out-of-flatness, may cause out-of-plane vibrations under live load. Studies have shown that larger initial out-of-flatness produces changing lateral deflection ("oil canning" or "vibrating") under live load, and, therefore, creates a potential for fatigue problems under certain conditions. Also, inspection of older existing bridges has revealed out-of-plane vibrations of webs under live load. Where Category E details (such as lateral bracing connections) are connected to the vibrating webs, stress ranges (determined from strain gages) were observed in one case to be as much as twice that which would occur if the webs were not vibrating, and estimated fatigue life was reduced by 75 to 90 percent of the fatigue life that would be expected without web vibration. The magnitude of oil canning varies from case-to-case.

Limited studies of existing structures suggest that web vibration will not be a serious problem if Category E details do not exist in the areas of web vibration; in that case category, C would determine fatigue life.

Additional studies are needed to confirm this. The potential for fatigue resulting from vibrations of girder webs under live load is a problem that may not have been adequately addressed in the LRFD Specification A6.10.6.3, which was intended to control fatigue due to the lateral deflection of girder webs under live load, may not adequately model in-service performance. This potential problem becomes more important in structures designed using more refined methods of analysis, because the conservative difference between the in-service stress range and the design stress range for conventional design may be significantly reduced by the refined method.

The γ_w factor in Equation 6.10.1.9.3P-1 is intended to provide for the potential reduction in fatigue life due to web vibrations which may be introduced as a result of web buckling during construction.

$$F_{cr} = \frac{k \pi^2 E}{12 (1 - \mu^2)} \left(\frac{t_w}{b} \right)^2 = \text{Critical Buckling Stress}$$

For $\mu = 0.3$ = Poisson's ratio for steel,

$k = 23.9$ (Case 5), and

$b = 2 D_c$

$$F_{cr} = \frac{5.4 E}{\left(\frac{D_c}{t_w} \right)^2} = f_{cw} \text{ when } \gamma_w = 1.0$$

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VALUES OF k FOR PLATE BUCKLING

Case 1

a/b	0.2	0.3	0.4	0.5	0.6	0.7	0.8
k	27.0	13.2	8.41	6.25	5.14	4.53	4.20
a/b	0.9	1.0	1.1	1.2	1.3	1.4	1.41
k	4.04	4.00	4.04	4.13	4.28	4.47	4.49

Case 2

a/b	0.4	0.5	0.6	0.7	0.8	0.9	1.0
k	9.44	7.69	7.05	7.00	7.29	7.83	7.69

Case 3

a/b	1.0	1.1	1.2	1.3	1.4	1.5	1.6
k	1.70	1.56	1.47	1.41	1.36	1.34	1.33
a/b	1.7	1.8	1.9	2.0	2.2	2.4	
k	1.33	1.34	1.36	1.38	1.45	1.47	

Case 4

a/b	0.5	1.0	1.2	1.4	1.6	1.8	2.0
k	4.40	1.44	1.14	0.952	0.835	0.755	0.698
a/b	2.5	3.0	4.0	5.0	For longer plates $k = 0.456 + (a/b)^2$		
k	0.610	0.504	0.516	0.506			

Case 5

a/b	0.4	0.5	0.6	0.67	0.75
k	29.1	25.6	24.1	23.9	24.1
a/b	0.8	0.9	1.0	1.33	1.5
k	24.4	25.6	25.6	23.9	24.1

Case 6*

a/b	1.0	1.2	1.4	1.6	1.8	2.0	3.0	5.0	∞
k	9.4	8.0	7.3	7.0	6.8	6.6	6.1	5.6	5.3

* b is always the short dimension for Case 6.

Figure C6.10.1.9.3P-1 – Formula for Plate Buckling

6.10.1.11P Lateral Support of Top Flanges Supporting Timber Decks

The compression flanges of girders supporting timber floors shall not be considered to be laterally supported by the flooring, unless the floor and fastenings are specially designed to provide such support. Laminated timber decks shall be provided with steel clips designed to furnish adequate lateral support to the top flange.

6.10.3 Constructibility**6.10.3.2 Flexure****6.10.3.2.4 Concrete Deck**

Delete A6.10.3.2.4.

C6.10.3.2.4

The following shall replace AC6.10.3.2.4.

The requirements of A6.10.3.2.4 are superseded by D6.10.1.7, which requires the area of deck longitudinal

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6.10.3.2.5P *Erection Analysis*

Evaluate lateral deflections in accordance with BD-620M.

6.10.3.2.5.1P *Slab Placement*

An analysis shall be performed to determine an acceptable slab placement sequence. The analysis shall address (but is not limited to) the following items:

- (a) Change in the stiffness in the girder as different segments of the slab are placed and as it affects both the temporary stresses and the potential for "locked-in" erection stresses
- (b) Bracing (or lack thereof) of the compression flange of girders and its effect on the stability and strength of the girder
- (c) Stability and strength of the girder through slab placement
- (d) Bracing of overhang deck forms
- (e) Uplift at bearings
- (f) Temperature changes as prescribed in D3.12.2.1.

The analysis of slab placement shall be done in an incremental fashion using a concrete modulus of elasticity equal to 70% of the concrete modulus elasticity at 28 days for concrete which is at least 24 hours old, assuming no retarder admixture is permitted. If retarder admixture is specified, it shall be indicated on the contract drawings, and the analysis shall be completed assuming 48 hours before gaining stiffness for lateral resistance. This means the stiffness of the model will change at the many different stages.

In no case shall the final design moment stresses or forces be less than those determined from an analysis in which the weight of the deck slab is applied all at once.

Slab concrete, which is less than 24 hours old (or 48 hours old when retarder is used), cannot be considered to provide lateral support for the embedded top flange of the girder. Conversely, slab concrete which is more than 24 hours old (or 48 hours old when retarder is used) can be considered to provide full lateral support for the embedded top flange of the girder. If the contractor can demonstrate that the concrete will provide lateral support for the embedded top flange in less than 24 hours (or 48 hours old when retarder is used), that limiting time may be used with the approval of the Chief Bridge Engineer.

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reinforcement to be at least 1 percent of the deck slab area.

C6.10.3.2.5.1P

During the mid-1980's, several of the Department's girder bridges experienced problems during placement of the slab. It is believed that bridges with highly unsymmetrical, deep steel girders combined with wide beam spacing and large overhang dimensions are more susceptible to problems during construction than are the typical earlier steel girder bridges which use more nearly symmetrical steel girders combined with closer beam spacing and smaller overhang dimensions. Since significant reduction in the construction cost of a bridge can be achieved by use of highly unsymmetrical, deep steel girders in conjunction with wide beam spacing and large overhang dimensions, an analysis must be performed to ensure that these types of girders provide adequate stability and strength through slab placement.

With skewed, curved, and/or continuous steel girder bridges, temporary uplift conditions at bearings can occur during the deck pour. Designers should evaluate the potential for uplift in bearings as part of the deck pour sequence evaluation. Designers should address temporary uplift conditions as follows:

- Where the temporary uplift is not detrimental to the long-term performance of the bearing, or does not result in adverse stability conditions, temporary uplift is permitted. In this case, the designer should identify in the construction plans the individual bearing locations where uplift is expected and during what stages of the deck pour the uplift will occur. A note stating that the uplift is temporary and permitted as part of construction should also be provided in the construction plans.
- Where uplift is determined to be unacceptable for individual bearing types or structure stability, the designer should identify in the construction plans the individual bearing locations where uplift is expected. Hold down forces and any other design requirements for restraining devices should be shown in the plans for the contractor's use in designing these components. Forces and design requirements for individual deck pour stages, as applicable, should be provided. The designer should verify the viability of at least one type of restraining device to meet the design requirements and provide schematic details of the device in the construction plans.

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From the results of the analysis of slab placement and lateral support conditions described above, the bending and shear strength of girder shall be checked.

6.10.3.2.5.2P Deck Slab Overhang Form Support

For the erection condition with the overhang form support system, the strength and stability of the fascia girder shall be ensured by applying the dead load of the overhang concrete and any construction equipment to the girder as follows:

- (a) The standard form support system, shown in Figure 6.10.3.2.5.2P-1, may be used where:
- (1) Girder web depth is less than 8'-0"
 - (2) Deck slab overhang is less than 4'-9"
 - (3) Slab thickness is equal to or less than 10 in.
 - (4) Transverse stiffener spacing does not exceed the depth of the girder
 - (5) In regions where γ_w (see D6.10.1.9.3P) is less than 2.5 and the factored dead load shear, using a load factor of 4.0, is less than the buckling shear given in A6.10.9.3.

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- The effects from temperature change on the curved and skewed bridges are mostly functions of the girder support conditions. When minimum required restraint necessary for girder(s) global stability (i.e., prevention of global buckling of the girder or collection of girders) is provided, the applied temperature change may not have an appreciable impact on overall bridge deflections and stresses.

C6.10.3.2.5.2P

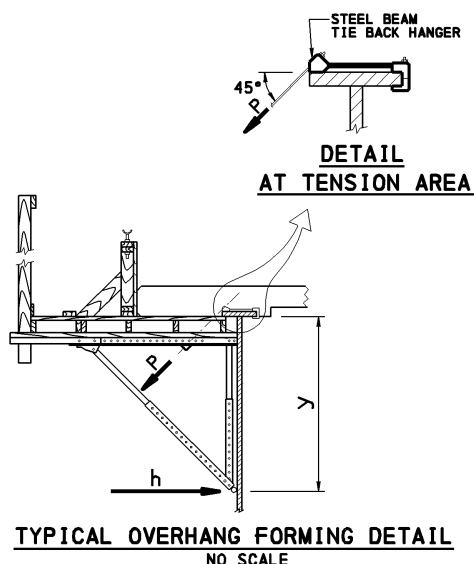
The requirements of this article can be met by reducing the length of some deck pours, or by increasing the size of the steel girder section, or by a combination of both. For original designs, the designer should obtain input from contractor and fabricators about the economics of those alternatives. Note, also, that only a relatively short length of the critical spans will be affected by the constructibility criterion.

The intent of the required checks is to control the buckling of the flanges and the webs of steel girders. It is felt that there is a potential for fatigue cracking if steel plates are allowed to buckle due to "oil-canning" effects.

The preferred upper limit on the deck slab overhang is 4'-0" considering factors such as deck forming and deck finishing.

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NOMINAL DEPTH y (in.)	MAXIMUM PERMISSIBLE HORIZONTAL LOAD h (kip/ft.)
36	1.25
56	0.75
76	0.54

MAXIMUM PERMISSIBLE JACK SPACING 4 FT.

The fascia girders are designed for a temporary construction load applied to the web at a maximum 4 ft. interval. This load (see table) approximates the horizontal component of a deck overhang form support bracket and consists of an allowance for the weight of the concrete, forms and incidental loads, plus the deck finishing machine. Where a transverse stiffener spacing, less than that required for the final design shear, is indicated for constructibility, the spacing for the final design shear may be used if the overhang forms are supported from the bottom flange of the fascia girder, or if the girder web is adequately braced to prevent buckling due to loads from web-bearing form support brackets. The contractor has the option to modify the overhang bracket from that described herein provided working drawings including calculations, sealed by a professional engineer licensed in the Commonwealth of Pennsylvania, are submitted for review and acceptance and show the modifications do not cause unacceptable deformations or stresses in the bridge and it is understood the contractor is ultimately responsible for the satisfactory completion of the bridge.

Figure 6.10.3.2.5.2P-1 - Typical Overhang Forming Detail and Note

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Where these requirements are satisfied, original designs of fascia girders shall provide transverse stiffeners throughout the span at a maximum spacing of D , including the region where stiffeners are not required for the final design shear or where a spacing larger than D would be satisfactory for the final design shear. This requirement ensures reasonable constructibility. The stiffener spacings required for both constructibility and final design shear shall be shown on the contract drawings (preferably on the girder elevations), and the sketch and note from Figure 6.10.3.2.5.2P-1 shall be included on the contract drawings.

- (b) For deck slab overhangs which do not meet the requirements of (a), the designer of the original structure shall review the condition with the Chief Bridge Engineer's office as part of the TS&L submission. If it is determined that web-supported overhang form brackets cannot be permitted, the following note shall be included in the general notes:

Support deck slab overhang forms from the bottom flange of the fascia girder, unless the girder web is adequately supported to prevent buckling due to loads from web-bearing form supports.

- (c) Contractor-designed alternates shall meet the requirements of this article. The stiffener spacing and a description of the deck overhang form support system, including the loads, shall be shown on the conceptual design drawings submitted for approval.
- (d) All DM-4 and appropriate AASHTO LRFD provisions in regard to flange and web buckling must be checked.
- (e) For additional criteria on exterior girder rotation due to large cantilever deck slabs, see D9.7.1.5.1P.

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For rolled beams spans, this is typically not a controlling design consideration with small deck overhangs less than 2 ft. overhangs.

The revision to this note was developed by an APC Subcommittee for Stability of Steel Bridge Superstructures. The note was modified to provide more flexibility to the contractor to use deck overhang form brackets that have nominal depths greater than the typical 3'-0" bracket depth. The maximum permissible horizontal load value was developed based on field measurements of steel bridges constructed in 1999 in District 5-0 with deck overhangs in the range of 4'-8" and a limited finite element analysis study of lateral web deflections of steel girders subjected to concentrated horizontal forces on the girder web.

Design modifications should consider web stress, overall web deformation, relative web deformation, the resulting deck overhang deflection, and the resulting effects on the finished deck profile. The contractor is responsible for selecting and providing calculations for the overhang forming system as required by Publication 408 Section 1050.3(c)2. Publication 408 Section 105.01 (c) specifies the responsibility of the work remains with the contractor regardless of reviews and/or acceptance of submitted working drawings by the Department.

Unacceptable deformations of the web or top flange results in deflection of the overhang bracket causing problematic deck finish and ride quality.

If an overhang is braced to within 6 inches of the bottom flange, it shall be considered braced to the bottom flange. Deck overhang forms for rolled beams spans, due to their shallow depth, are typically supported in this manner.

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6.10.3.2.5.3P Deck Slab Overhang Rotation

The designer shall consider the effects of out-of-plane girder rotations, common with skewed bridges, on deck elevations.

C6.10.3.2.5.3P

Out-of-plane girder rotations will cause the overhang formwork to also rotate. In an increasing magnitude from the web of the fascia girder to the outside edge of the formwork, the formwork will move upward or downward, depending on the direction of rotation, during the deck pour. It may be desirable to pre-rotate the overhang formwork so that the as-designed deck overhang cross slope is obtained after the deck pour is complete. Additionally, it may be desirable to relocate the deck finishing machine support railing from its typical position on the overhang formwork to the fascia girders. This will minimize the upward and downward movements of the finishing machine during the deck pour due to out-of-plane girder rotations. Hand finishing work will be necessary for the deck area beyond the limits of the finishing machine.

The designer may consider approximating the anticipated girder rotation based on girder differential vertical displacements.

6.10.3.5 Dead Load Deflection

The following shall replace A6.10.3.5.

The provisions of A6.7.2 and D6.7.2 shall apply, as applicable.

6.10.4 Service Limit State**6.10.4.2 Permanent Deformations***6.10.4.2.1 General*

The following shall replace the bulleted list of A6.10.4.2.1.

- For sections that are composite for positive flexure, the section assumed in calculating the stresses shall be as specified in A6.10.1.1.1b and D6.10.1.1.1b.
- For sections that are composite for negative flexure, the section assumed in calculating the stresses shall be as specified in D6.10.1.1.1c.
- For sections that are noncomposite, the properties of the steel section alone shall be used for calculation of the flexural stresses in the structural steel.

C6.10.4.2.1

The following shall replace the second and third paragraphs of A6.10.4.2.1.

New bridges are required to be composite along their entire length. Existing bridges may be noncomposite along their entire length or only composite in the positive moment regions. The requirements of the third bullet in D6.10.4.2.1 is meant for use in evaluating noncomposite sections in positive and/or negative flexure in such existing bridges.

6.10.4.2.2 Flexure

Delete the second paragraph of A6.10.4.2.2

C6.10.4.2.2

Delete the ninth and the last paragraphs of AC6.10.4.2.2.

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6.10.5 Fatigue and Fracture Limit State**6.10.5.3 Special Fatigue Requirement for Webs**

The following shall replace the first paragraph of A6.10.5.3

For the purposes of this article, the factored fatigue load shall be determined using the Fatigue I load combination specified in Tables A3.4.1-1 and in D3.4.1.1P, with the fatigue live load taken as specified in A3.6.1.4 and D3.6.1.4, multiplied by the Pennsylvania Traffic Factor (PTF). The PTF is specified in D6.6.1.2.2.

6.10.6 Strength Limit State**6.10.6.2 Flexure***6.10.6.2.3 Composite Sections in Negative Flexure and Noncomposite Sections*

Delete the last paragraph of A6.10.6.2.3.

6.10.7 Flexural Resistance—Composite Sections in Positive Flexure**6.10.7.1 Compact Sections***6.10.7.1.2 Nominal Flexural Resistance*

Delete the remaining portion of A6.10.7.1.2 starting after the definition of R_p .

6.10.8 Flexural Resistance - Composite Sections in Negative Flexure and Noncomposite Sections**6.10.8.2 Compression-Flange Flexural Resistance***6.10.8.2.3 Lateral Torsional Buckling Resistance**C6.10.6.2.3*

Delete the last paragraph of AC6.10.6.2.3.

C6.10.7.1.2

Delete the fourth through the eighth paragraphs of AC6.10.7.1.2.

C6.10.8.2.3

The following shall replace the sixth paragraph of AC6.10.8.2.3.

If $D_c t_w / b_f t_{fc}$ in Eq. A6.10.8.2.3-9 is taken as a representative value of 2.0, this equation reduces to $0.22b_{fc}$. Based on this assumption and $F_{yc} = 50$ ksi, the compact bracing limit is $L_p = 5.4b_{fc}$ and the noncompact bracing limit given by Eq. A6.10.8.2.3-5 simplifies to $L_r = 20b_{fc}$. Based on these same assumptions, the equation of AASHTO Appendix D, Section D6.4 gives corresponding limits on L_b that are generally larger than $5.4b_{fc}$. The limit given in AASHTO Appendix D, Section D6.4 is sufficient to develop F_{max} or M_{max} shown in Figure AC6.10.8.2.1-1 in cases involving a moment gradient along the unbraced length for which $C_b > 1.0$.

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6.10.9 Shear Resistance**6.10.9.1 General**

The following shall replace the first bulleted item of the third paragraph of A6.10.9.1.

- without a longitudinal stiffener and with a transverse stiffener spacing not exceeding $1.5D$, or

The following shall replace the fourth paragraph of A6.10.9.1.

Provisions for end panels shall be as specified in A6.10.9.3.3 and D6.10.9.3.3.

The following shall supplement A6.10.9.1.

Transverse stiffener spacing shall also satisfy the requirements of D6.10.3.2.5.2P for deck slab overhang form support.

6.10.9.3 Nominal Resistance of Stiffened Webs*6.10.9.3.3 End Panels*

The following shall replace the last paragraph of A6.10.9.3.3.

The transverse stiffener spacing for end panels without a longitudinal stiffener shall not exceed $0.5D$. The transverse stiffener spacing of end panels with a longitudinal stiffener shall not exceed 0.5 times the maximum subpanel depth.

6.10.10 Shear Connectors**6.10.10.1 General**

The following shall replace the third, fourth and fifth paragraphs of A6.10.10.1.

Shear connectors are required along the entire length of the girder for both straight and curved bridges when a composite girder analysis has been performed.

6.10.10.1.1 Types

The following shall supplement A6.10.10.1.1.

The minimum diameter of studs shall be $3/4$ in..

6.10.10.1.2 Pitch

The following shall replace the last paragraph of A6.10.10.1.2.

The center-to-center pitch of stud shear connectors shall

C6.10.10.1

The following shall supplement AC6.10.10.1.

Mechanical shear connectors provide for the horizontal shear at the interface between the concrete slab and the steel girder in the positive moment regions and the horizontal shear between the longitudinal reinforcement steel within the effective flange width and the steel girder in the negative moment regions.

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not exceed 24 in. and shall not be less than six stud diameters. The center-to-center pitch of channel shear connectors shall not exceed 24 in. and shall not be less than 6 in..

6.10.10.1.3 Transverse Spacing

The following shall supplement A6.10.10.1.3.

The minimum number of studs in a group shall consist of two in a single transverse row.

*6.10.10.1.4 Cover and Penetration**6.10.10.1.5P Splice Locations*

Shear connectors at splice locations shall be arranged to clear fasteners and shall be welded to the splice plate. Up to 20% fewer connectors, than required by design, are acceptable in the splice zone, provided that the deleted connectors are furnished as additional connectors adjacent to the splice.

6.10.10.2 Fatigue Resistance

The following shall replace the first bullet of A6.10.10.2.

- Where the projected 100-year single lane Average Daily Truck Traffic ($ADTT_{SL}$) is greater than or equal to 720 trucks per day, the Fatigue I load combination shall be used and the fatigue shear resistance for infinite life shall be taken as:

The following shall replace the third bullet of A6.10.10.2.

- Where the projected 100-year single lane Average Daily Truck Traffic ($ADTT_{SL}$) is greater than or equal to 1385 trucks per day, the Fatigue I load combination shall be used and the fatigue shear resistance for infinite life shall be taken as:

6.10.10.3 Special Requirements for Points of Permanent Load Contraflexure

The following shall supplement A6.10.10.3.

The provisions of this article are only applicable when analyzing existing bridges where the shear connectors are

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C6.10.10.1.4

The following shall replace the second sentence of AC6.10.10.1.4.

For plan presentation, show cover and penetration limits; do not detail stud height (see Standard Drawing BC-753M). Stud heights are determined in the field based on actual girder elevations.

C6.10.10.2

The following shall supplement AC6.6.1.2.3

PennDOT's design life is considered to be 100 years. In the overall development of the AASHTO LRFD Specification, the design life has been considered to be 75 years. The provisions of this section have been revised to reflect a 100 year design life.

C6.10.10.3

The following shall supplement AC6.10.10.3.

For new designs, PennDOT requires composite girders to have shear connectors along the full length of the girders

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not provided along the full length of the girder.

6.10.11 Stiffeners**6.10.11.1 Transverse Stiffeners***6.10.11.1.1 General*

The following shall supplement A6.10.11.1.1.

Transverse stiffeners shall also satisfy the requirements given in Standard Drawing BC-753M.

6.10.11.2 Bearing Stiffeners*6.10.11.2.1 General*

The following shall supplement A6.10.11.2.1.

Under full dead load, beam ends and all bearing stiffeners, including bearing stiffeners at piers, are to be vertical.

6.10.11.4P Stiffeners in Rigid-Frame Knees*6.10.11.4.1P Stiffener Spacing*

The spacing of stiffeners in rigid-frame knees shall satisfy both of the following equations:

$$f_a \leq F_{yc} - f_{cs} \quad (6.10.11.4.1P-1)$$

and

$$f_b \leq F_{yc} \quad (6.10.11.4.1P-2)$$

for which:

$$f_a = f_{cs} \left(\frac{3b^2}{Rt} \right) \left(\frac{\beta^2}{4 + 1.14\beta^4} \right) \quad (6.10.11.4.1P-3)$$

$$f_b = f_{cs} \left(\frac{3b^2}{Rt} \right) \left(\frac{\beta^3}{3.2 + \beta^3} \right) \quad (6.10.11.4.1P-4)$$

$$\beta = \frac{a}{b} \quad (6.10.11.4.1P-5)$$

where:

a = spacing of stiffeners (in.)

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and the provisions of this article will not be applicable.

The purpose of the additional connectors is to develop the reinforcing bars used as part of the negative flexural section.

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- b = half of flange width (in.)
- F_{yc} = specified minimum yield strength of a compression flange (ksi)
- f_{cs} = maximum compression Service I load flange stress (ksi)
- R = radius of flange curvature (in.)
- t = thickness of flange (in.)

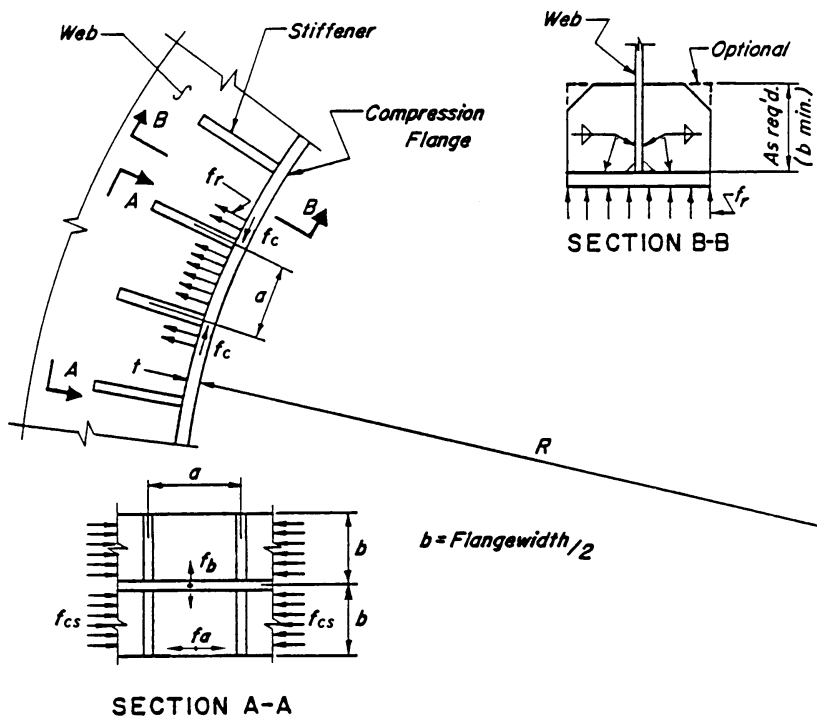


Figure 6.10.11.4.1P-1 - Stiffeners in Rigid-Frame Knees

6.10.11.4.2P Stiffener Design

The factored bearing resistance of stiffeners in rigid-frame knees, taken as specified in A6.10.11.2.3, shall be greater than P_b , taken as:

$$P_b = f_r ab \quad (6.10.11.4.2P-1)$$

for which:

$$f_r = \frac{f_c t}{R} \quad (6.10.11.4.2P-2)$$

where:

- a = spacing of stiffeners (in.)

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- b = half of flange width (in.)
- f_c = maximum factored compression flange stress (ksi)
- R = radius of flange curvature (in.)
- t = thickness of flange (in.)

6.10.12 Cover Plates**6.10.12.3P Cover Plate Length and Width**

The length of any welded cover plate added to a rolled beam shall extend the full-length of the rolled beam, including the bearing area, or the full-length of the rolled beam field section in the case of a spliced beam unless otherwise approved by the Chief Bridge Engineer. The use of partial length cover plates is allowed for rehabilitation projects with detailed fatigue analysis. Partial length cover plates must have bolted connections at the ends.

The width of the plate shall not exceed the width of the flange by more than 6 in., or six times the thickness of the cover plate, whichever is less. Bottom flange cover plates preferably shall be wider than the bottom flange. Top flange cover plates shall be of constant width, preferably narrower than the top flange. When a cover plate narrower than the flange is used, the width of the plate shall be at least 2 in. less than the width of the flange. The width of a cover plate connected by fillet welds shall be no greater than 24 times the plate thickness.

6.11 BOX-SECTION FLEXURAL MEMBERS**6.11.1 General**

The following shall supplement A6.11.1

If girders deeper than 14 ft. are necessary for economical design or esthetic considerations, they shall be designed with a horizontal field splice. The approval of the Chief Bridge Engineer shall be required when a horizontal field splice is incorporated. The contract plans shall include a statement to permit the elimination of the horizontal field splice at the Contractor's option.

6.11.4 Service Limit State

Delete the last paragraph of A6.11.4.

6.11.6 Strength Limit State**6.11.6.2 Flexure****C6.10.12.3P**

The Department does not allow partial length cover plates for new designs.

C6.11.1

The following shall supplement AC6.11.1.

Flanged web splices (i.e., those constructed with angles) are not desirable.

C6.11.4

Delete the last paragraph of AC6.11.4.

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6.11.6.2.3 *Sections in Negative Flexure*

C6.11.6.2.3

Delete the second sentence of the first paragraph of A6.11.6.2.3

Delete the second paragraph of AC6.11.6.2.

6.11.7 Flexural Resistance - Sections in Positive Flexure**6.11.7.1. Compact Sections**6.11.7.1.2 *Nominal Flexural Resistance*

C6.11.7.1.2

Delete the last sentence of the second paragraph of AC6.11.7.1.2.

6.13 CONNECTIONS AND SPLICES**6.13.1 General****C6.13.1**

The following shall replace the first paragraph of A6.13.1.

The following shall supplement AC6.13.1.

Except as specified otherwise, connections and splices for main members (both flanges and webs) shall be designed at the strength limit state for not less than the larger of:

Often stresses on the top flange of a composite girder are low under design loads. These criteria would require a top splice plate to be about 75% of the area of the top flange, and have the appropriate number of bolts.

- The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point
- 75 percent of the factored flexural, shear, or axial resistance of the member or element

The following shall replace the sixth paragraph of A6.13.1

Only standard-size bolt holes shall be used in connections in horizontally curved bridges including for connecting diaphragms and cross frames.

The following shall supplement A6.13.1.

If it is necessary to cope a flange in order to provide clearance at the end connection of a floorbeam or stringer, the bending resistance of the member at the cope location shall not be decreased by more than 50%. No sharp notches shall be introduced as a result of coping. The maximum practical radius shall be maintained at all copes with an absolute minimum radius of 2 in.

6.13.2 Bolted Connections**6.13.2.1 General****C6.13.2.1P**

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The following shall replace the second paragraph of A6.13.2.1.

High-strength bolted joints shall be designated as slip-critical connections. Bearing-type connections may be used on rehabilitation projects if approved by the Chief Bridge Engineer.

6.13.2.4 Holes*6.13.2.4.1 Types**6.13.2.4.1b Oversize Holes*

The following shall replace A6.13.2.4.1b.

Approval of Chief Bridge Engineer must be obtained before oversize holes can be used in any or all plies of slip-critical connections. Oversize holes are not permitted in diaphragms or cross frames of curved girder bridges. Oversize holes shall not be used in bearing-type connections.

*6.13.2.4.1c Short-Slotted Holes***6.13.2.5 Size of Bolts**

The following shall supplement A6.13.2.5.

Fasteners shall be of the size shown on the contract plans, but generally shall be 7/8 in. in diameter.

6.13.2.6 Spacing of Bolts*6.13.2.6.1 Minimum Spacing and Clear Distance*

COMMENTARY

When detailing bolted connections, tightening clearance between flange and web bolts need to be taken into account. Manual of Steel Construction, American Institute of Steel Construction, provides information on assembling clearances for threaded fasteners which can be used to avoid bolt interference problems.

C6.13.2.4.1cP

On skew bridges, short slotted holes for cross-frame connections should be utilized to allow adjustments for temporary and permanent conditions.

C6.13.2.5P

Typically, high-strength bolts will be 7/8 in. diameter mechanically galvanized AASHTO M 164 (ASTM A 325) bolts. This is the typical high-strength bolt used in the past.

C6.13.2.6.1

The following shall supplement AC6.13.2.6.1.

The preferred distance between centers of bolts in standard holes shall not be less than the values in Table C6.13.2.6.1P-1:

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Table C6.13.2.6.1P-1 - Preferred Bolt Spacing

Diameter Bolt (in.)	Preferred Distance between Centers of Bolts (in.)
5/8	2 1/4
3/4	2 1/2
7/8	3
1	3 1/2

6.13.2.7 Shear Resistance**C6.13.2.7**

The following shall supplement the first paragraph of AC6.13.2.7.

For steel plate girder flange splices greater than 50 in. in length, the 20% reduction shall be applied to the nominal shear resistance of a bolt, calculated using Equation A6.13.2.7-1 and A6.13.2.7-2, because the axial force is parallel to the line of bolts. In such flange splices, the 50 in. length is to be measured between the extreme bolts on only one side of the connection. The 20 percent reduction should not be applied for web bolts subjected to shear and moment.

6.13.2.8 Slip Resistance**C6.13.2.8**

The following shall replace the first bulleted item in the second paragraph of A6.13.2.8 (the definition of Class A surface).

- Class A surface: blast cleaned surfaces with Class A coatings

The following shall supplement A6.13.2.8.

For values of K_s , use Class A surface conditions for design, unless a paint is tested and proven to conform to Class B conditions. If Class B is used, field testing and controls must be specified in the contract drawings or construction specifications.

Delete the fourth paragraph in A6.13.2.8 which starts "The contract document shall specify that joints having..." and replace it with the following.

The following note shall be placed on the contact drawings:

"Blast clean the faying surfaces of splices and connections of all structural elements in accordance with Publication 408 Section 1060.3(b)3. Reblast unpainted elements that remain unassembled for a period of 12 months or more following the initial cleaning."

The following shall supplement AC6.13.2.8.

The revision to the definition of Class A and the requirement to blast clean all faying surfaces is based on results of research conducted jointly by the University of Texas at Austin and the FHWA in the early 1980's on weathering steel connections. An extensive testing program conducted in conjunction with the research showed that weathering steel connections with a mill scale surface had an average slip coefficient, K_s , less than the 0.33 value for Class A. Blast cleaned weathering steel achieved an average slip coefficient above the 0.50 value specified for a Class B contact surface. The testing program incorporated a wide range of variables, including exposure of test specimens to an open environment for periods up to 12 months.

The UT/FHWA research suggests that present LRFD design policy, which allows a mill scale surface for Class A, could result in weathering steel connections that do not meet the slip coefficient value for a Class A contact surface. The revision to the definition of Class A and the requirements to blast clean all faying surfaces will add desired safety into Department projects.

Inherent factors of safety in the design of connections should ensure the serviceability of in-place weathering steel structures where the slip critical condition controlled the design.

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Designers are directed to review Bulletin 15 (Publication 35) for current paint systems and corresponding slip coefficients.

6.13.3 Welded Connections**6.13.3.1 General**

The following shall supplement A6.13.3.1.

Field welding is generally prohibited. Provisions may be made for attachment of stay-in-place forms, bearing plates and sole plates of pot bearings (but not the pot bearing itself). All areas where field welding is permitted shall be specifically designated on the contract plans. The fatigue provisions of this specification shall apply to the design of all affected members.

The regions of welded structures requiring non-destructive testing (NDT), along with the allowable types of NDT, shall be shown on the contract plans.

6.13.3.8P Intersecting Welds

Intersecting welds which provide a potential crack path into the web or flange of a girder from an attachment will not be permitted. The termination of the fillet weld to prevent the intersection shall provide a minimum clearance of 1 1/2 in., unless another clearance is required by other design documents. Transverse groove welds shall not be terminated to prevent the intersection.

6.13.3.9P Intermittent Fillet Welds

Intermittent fillet welds are prohibited, unless they are incorporated in the final weld in accordance with AASHTO/AWS Bridge Welding Code.

6.13.3.10P Minimum Edge Distance

A minimum edge distance of 1 in. shall be maintained from a fillet weld termination to the edge of a base metal plate in the direction of the weld.

C6.13.3.1

The following shall supplement AC6.13.3.1

The AASHTO/AWS D1.5M/D1.5 Bridge Welding Code describes the appropriate application of the types of NDT.

Use the AASHTO/AWS D1.1M/D1.1 Structural Welding Code for the welding of new tubular structures, pipes, piles and existing steel which are not covered by AASHTO/AWS D1.5M/D1.5 in accordance with Publication 408, Section 1105.

C6.13.3.10P

An example of minimum edge distance is graphically shown in Figure C6.13.3.10P-1.

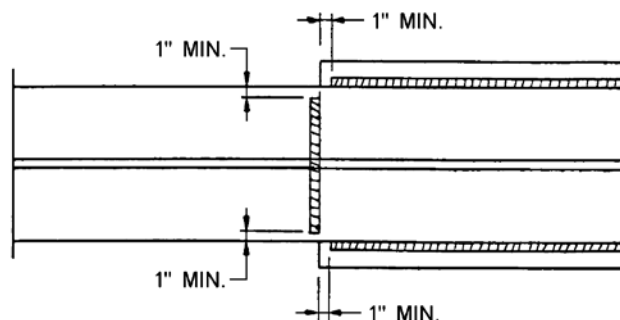


Figure C6.13.3.10P-1 - Minimum Edge Distance

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6.13.6 Splices**6.13.6.1 Bolted Splices***6.13.6.1.1 General*

The following shall replace A6.13.6.1.1.

Bolted splices shall be designed at the strength limit state to satisfy the requirements specified in A6.13.1 and D6.13.1. Where a section changes at a splice, the smaller of the two connected sections shall be used in the design. Develop bolted field splices for steel beams and girders in accordance with Standard Drawing BD-616M. Splices shall be designed using the Department's SPLRFD program.

*6.13.6.1.4 Flexural Members**6.13.6.1.4a General*

The following shall supplement A6.13.6.1.4a for straight girders.

Splices subjected to tension shall satisfy the requirements specified in A6.13.5.2.

6.13.6.1.4b Web Splices

The following shall supplement A6.13.6.1.4b.

The elastic methods shall be used to determine the shear force in splice bolts. The ultimate strength method is not permitted.

6.13.6.1.4c Flange Splices

The following shall supplement A6.13.6.1.4c.

For bolted flexural members, bolted splices in flange parts should not be used between field splices, unless approved by the Chief Bridge Engineer. In any one flange, not more than one part should be spliced at the same cross-section. If practicable, splices should be located at points where there is an excess of section.

6.13.6.1.5 Fillers

The following shall replace the third paragraph of A6.13.6.1.5.

Fillers 1/4 in. or more in thickness shall consist of not more than two plates, unless approved by the Chief Bridge Engineer.

C6.13.6.1.4a

Delete the first paragraph of AC6.13.6.1.4a.

C6.13.6.1.4b

The following shall supplement AC6.13.6.1.4b for straight girders.

For bolt groups subjected to eccentric shear, a traditional conservative approach is often used in which the bolt group is treated as an elastic cross-section subjected to direct shear and torsion. A vector analysis is performed assuming that there is no friction and that the plates are rigid and the bolts are elastic.

C6.13.6.1.4c

The following shall supplement A6.13.6.1.4c for straight girders.

For compression, use an unbraced length equal to zero, making $P_n = F_y A_s$

SPECIFICATIONS

The following shall replace the fifth paragraph of A6.13.6.1.5.

The specified minimum yield strength of fillers 1/4 in. or greater in thickness shall not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 36 ksi unless approved by the Chief Bridge Engineer.

6.13.6.2 Welded Splices

The following shall replace the third paragraph of A6.13.6.2.

Welded field splices shall not be used without written approval of the Chief Bridge Engineer.

6.14 PROVISIONS FOR STRUCTURE TYPES**6.14.2 Trusses****6.14.2.2 Truss Members****6.15 PILES****6.15.1 General**

The following shall replace A 6.15.1.

Piles shall be designed as structural members capable of safely supporting all imposed loads.

For a pile group composed of only vertical piles which is subjected to lateral load, the pile structural analysis shall include explicit consideration of soil-structure interaction effects using a COM624P analysis (Wang and Reese, 1993) or LPILE 5.0 analysis.

Based on the parametric study conducted by the Department, which is described in the commentary, an abutment or retaining wall with a pile group composed of both vertical and battered piles which is subjected to lateral load shall be designed assuming that all lateral load is resisted by the horizontal component of the axial capacity of the battered piles. The vertical load shall be distributed among piles in the group using a simple elastic procedure. The use of the above design procedure with any of the following conditions requires the approval of the Chief Bridge Engineer:

- Piles with a specified steel yield strength other than 36 ksi

COMMENTARY

C6.13.6.2

The following shall supplement AC6.13.6.2.

Use the AASHTO/AWS D1.1M/D1.1 Structural Welding Code for the welding of new tubular structures, pipes, piles, and existing steel, which are not covered by AASHTO/AWS D1.5M/D1.5 in accordance with Publication 408, Section 1105.

C6.14.2.2

The following shall supplement AC6.14.2.2.

Truss members should be evaluated for wind-induced vibrations in accordance with DC4.7.2.2.3.

C6.15.1

The following shall replace AC6.15.1.

To develop the recommended distribution of lateral load among piles supporting a typical bridge abutment, a parametric study (Kelly, et al, 1995) was performed using the program GROUP (Reese, et al, 1994). A second purpose of this parametric study was to determine if the Department's lateral deformation criteria of 1/2 in. for the service limit state and 1 in. for the strength limit state were satisfied. These criteria were met for all analyses representative of the Department's practice. The variables evaluated in the parametric study included:

- HP12X53 and HP10X42 piles with about four-pile diameters center-to-center spacing
- End bearing piles on rock driven through a medium stiff-to-stiff clay deposit, and friction piles in deposits of medium dense and loose sand
- Pile lengths of 10, 30, and 50 ft.
- Vertical and horizontal load levels consistent with

SPECIFICATIONS

- Piles with bending stiffness properties less than HP10X42
- Very soft clays or very loose sands as defined in Standard Drawing BC-795M
- Piles with bending stiffness properties less than HP12X53 in soft clays or loose sands as defined in Standard Drawing BC-795M
- Vertical load to horizontal load ratio less than 3.5 at the service limit state

COMMENTARY

common Department designs

- Pile-head fixity conditions of fixed, pinned and partially fixed

For typical pile groups containing battered piles designed using the simplified procedure, the pile study indicates that:

- Combined stresses in upper portions of the battered and vertical piles due to axial load and bending are generally less than 11.9 to 13.8 ksi, which is consistent with previous practice
- The fraction of the total lateral load resisted by bending of the vertical piles is generally less than about 20 percent
- The check of structural pile capacity for combined axial load and flexure in the upper portion of the pile using the LRFD Interaction Equations in A6.9.2.2 does not control the pile design
- Lateral deflections are well below acceptable magnitudes
- As pile stiffness increases, horizontal deformations and associated bending stresses decrease such that the simplified method remains applicable

In cases for which pile and soil conditions differ significantly from those conditions examined in the parametric study, a suitable analysis should be performed which incorporates the necessary soil-structure interaction factors. This analysis may comprise finite element analysis, p-y analysis, or other applicable methods.

Lateral deflections and maximum bending stresses for laterally-loaded pile groups generally occur within a depth below the pile cap equal to approximately 10 pile diameters. Therefore, the presence of poor material (very soft clays or very loose sands) within the upper 10 pile diameters invalidates use of the simplified method due to the potential for pile overstressing and excessive deformations under lateral loads. If these conditions exist, the designer may consider the following options:

- Improve in place or remove and replace the poor material. These may be viable options when the thickness of poor material is small and close to the ground surface
- Perform a more rigorous, problem specific analysis to define pile stress levels and pile group deformations. This type of analysis may be performed using software such as GROUP, Reese, et al (1994)

SPECIFICATIONS

6.15.2 Structural Resistance

The following shall replace A 6.15.2.

Resistance factors, ϕ , for the strength limit state shall be taken as specified in D6.5.4.2. The resistance factors for axial resistance of piles in compression which are subject to damage due to driving shall be applied only to that section of the pile likely to experience damage. Therefore, the specified ϕ factors of 0.35 and 0.45, specified in D6.5.4.2 for piles subject to damage, shall be applied only to the axial capacity of the pile. In addition, the ϕ factors of 0.60 and 0.85, specified in D6.5.4.2 for axial and flexural resistance, respectively, of undamaged piles, shall be applied to the combined axial and flexural resistance of the pile in the interaction equation for the compression and flexure terms respectively. This design approach is illustrated on Figure 6.15.2-1P.

For piles bearing on soluble bedrock, the ϕ factor of 0.25 shall be applied to the axial capacity of the pile using a pile yield strength $F_y = 36$ ksi.

COMMENTARY

A thin seam or lens of poor material below the upper 10 pile diameters will not typically affect the applicability of the simplified method.

C6.15.2

The following shall replace A 6.15.2.

Due to the nature of pile driving, additional factors must be considered in selection of resistance factors that are not normally accounted for in steel members. The factors considered in development of the specified resistance factors include:

- Unintended eccentricity of applied load about expected point of application
- Variations in material properties of pile
- Pile damage due to driving

These factors are discussed by Davisson (1983). While the resistance factors specified herein generally conform to the recommendations given by Davisson (1983), they have been modified to reflect the common practice of the Department. Specifically, the resistance factors presented in D6.5.4.2 have been selected in a manner such that, when combined with an average load factor of 1.45, the equivalent factor of safety calculated as the ratio of the appropriate load to resistance factor is comparable to the factor of safety previously used by the Department.

The factored compressive resistance, P_r , includes reduction factors for unintended load eccentricity and material property variations, as well as a reduction for potential damage to piles due to driving, which is most likely to occur near the tip of the pile. The resistance factors for computation of the factored axial pile capacity near the tip of the pile are 0.35 and 0.45 for severe and good driving conditions, respectively. The ϕ factor of 0.25 for piles bearing on soluble bedrock is intended to safeguard against the potential of the loss of geotechnical capacity in soluble bedrock.

For steel piles, flexure occurs primarily toward the head of the pile. This upper zone of the pile is less likely to experience damage due to driving. Therefore, relative to combined axial compression and flexure, the resistance factor for axial resistance ($\phi_c = 0.60$) accounts for both unintended load eccentricity and pile material property variations, whereas the resistance factor for flexural resistance ($\phi_f = 0.85$) accounts only for variations in pile material properties.

Typically, due to the lack of a detailed soil-structure interaction analysis of pile groups containing both vertical and battered piles, evaluation of combined axial and flexural loading will only be applied to pile groups containing no battered piles.

SPECIFICATIONS

COMMENTARY

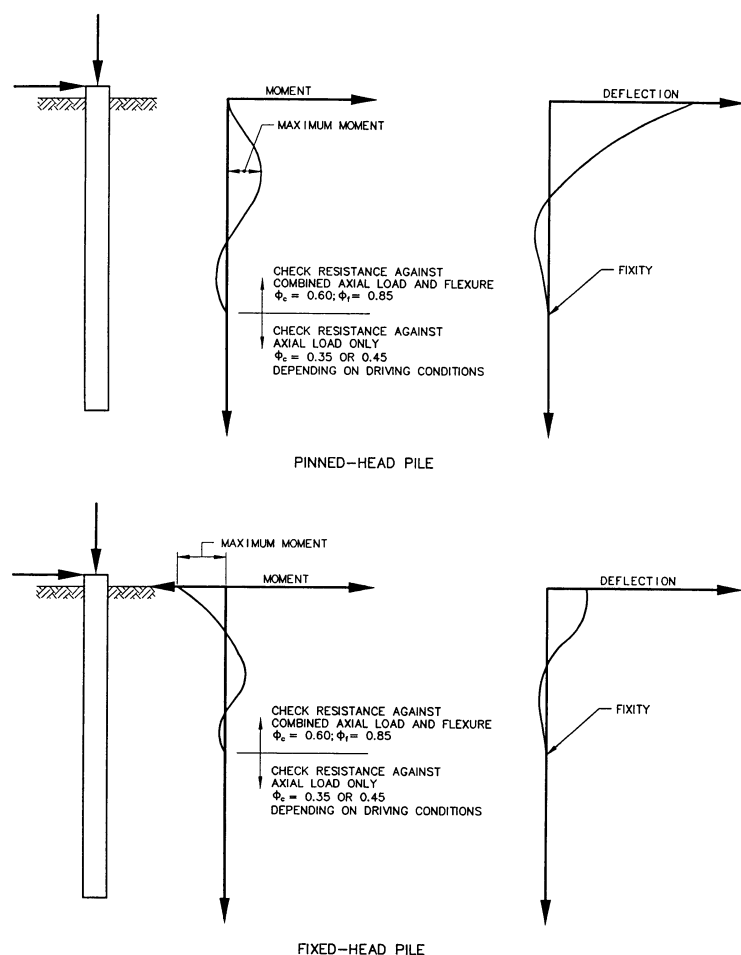


Figure 6.15.2-1P - Distribution of Moment and Deflection in Vertical Piles Subjected to Lateral Load

6.15.3 Compressive Resistance

The following shall supplement A6.15.3.

The design of steel piles shall follow A6.9, except as specified herein.

6.15.3.1 Axial Compression

The following shall replace A 6.15.3.1.

For piles under axial load, the factored resistance of piles in compression, P_r , shall be taken as specified in A6.9.2.1 using the resistance factor, ϕ_c , specified in D6.5.4.2.

6.15.3.2 Combined Axial Compression and Flexure

C6.15.3.2P

The following shall replace A 6.15.3.2.

Piles subjected to axial load and flexure shall be designed in accordance with A6.9.2.2 using the resistance factors, ϕ_c and ϕ_t , specified in D6.5.4.2.

SPECIFICATIONS

Vertical H-pile foundations designed using COM624P or LPILE 5.0 per D10.7.3.12.2P may use the values given in Tables 6.15.3.2P-1 and 6.15.3.2P-2.

where:

D	=	Depth of the pile (in.)
Area	=	Area of the pile (in. ²)
I_x, I_y	=	Moment of inertia about their respective axis (in. ⁴)
P_{uSERV}	=	COM624P or LPILE 5.0 pile load equivalent to pile resistance at Service Limit State (kips)
P_{uSTR}	=	COM624P or LPILE 5.0 pile load equivalent to pile resistance under severe driving conditions as defined in D6.15.2 (kips)
P_r	=	Factored axial resistance for combined axial and flexural resistance (kips)
M_{rx}, M_{ry}	=	Factored flexural resistance of the vertical pile in the x-axis and y-axis, respectively (kip-ft.)

COMMENTARY

The factored flexural resistance, M_{rx} , is based on either the plastic or elastic moment of pile considering web and compression flange slenderness requirements. The factored flexural resistance, M_{ry} , is based on the plastic moment per AC6.12.2.2.1.

For these tables the piles are considered as braced. If very weak soils, scour or voids are expected, the buckling requirements of D6.15.3.3 must be considered and the values shown in Tables 6.15.3.2P-1 and 6.15.3.2P-2 are not applicable.

P_{uSERV} and P_{uSTR} in Tables 6.15.3.2P-1 and 6.15.3.2P-2 are based on $0.25F_y A_s$ and $0.35F_y A_s$, respectively. For piles bearing on soluble rock (limestone, etc.) values for P_{uSERV} and P_{uSTR} equal to $0.16F_y A_s$ and $0.25F_y A_s$, respectively, are to be considered.

The section properties provided are for use in the COM624P or LPILE 5.0 analysis. The combined axial compression and flexural requirements of A6.9.2.2 shall be evaluated considering the results of the COM624P or LPILE 5.0 analysis and the resistances provided in the Tables 6.15.3.2P-1 and 6.15.3.2P-2.

The section properties may also be used in PennDOT's Integral Abutment Spreadsheet. However; since the capacities in the tables do not consider unbraced length, the structural pile capacity of vertical piles used in an Integral Abutment must be checked in accordance with DM-4 Appendix G using the Integral Abutment Spreadsheet.

SPECIFICATIONS

COMMENTARY

Table 6.15.3.2P-1 – Pile Properties, Factored Axial and Flexural Resistances with Full Pile Section. $F_y = 36$ ksi

HP	Depth D (in.)	Area (in. ²)	I_x (in. ⁴)	I_y (in. ⁴)	P_{uSERV} (kips)	P_{uSTR} (kips)	P_r (kips)	M_{rx} (kip-ft.)	M_{ry} (kip-ft.)
14 x 117	14.21	34.4	1220	443	310	434	743	495	228
14 x 102	14.01	30.0	1050	380	270	378	648	431	197
14 x 89	13.83	26.1	904	326	235	329	564	334	169
14 x 73	13.61	21.4	729	261	193	270	462	273	137
12 x 84	12.28	24.6	650	213	221	310	531	306	132
12 x 74	12.13	21.8	569	186	196	275	471	268	116
12 x 63	11.94	18.4	472	153	166	232	397	202	97
12 x 53	11.78	15.5	393	127	140	195	335	170	81
10 x 57	9.99	16.8	294	101	151	212	363	170	75
10 x 42	9.70	12.4	210	71.7	112	156	268	111	54

Table 6.15.3.2P-2 – Pile Properties, Factored Axial and Flexural Resistance with 1/16" Section Loss. $F_y = 36$ ksi

HP	Depth D (in.)	Area (in. ²)	I_x (in. ⁴)	I_y (in. ⁴)	P_{uSERV} (kips)	P_{uSTR} (kips)	P_r (kips)	M_{rx} (kip-ft.)	M_{ry} (kip-ft.)
14 x 117	14.09	28.73	1019	365	259	362	620	369	189
14 x 102	13.89	24.39	853	305	219	307	527	313	159
14 x 89	13.71	20.51	708	253	185	258	443	263	133
14 x 73	13.49	15.83	537	192	142	199	342	203	101
12 x 84	12.16	19.81	521	168	178	250	428	219	106
12 x 74	12.01	17.02	443	143	153	215	368	188	90
12 x 63	11.82	13.66	349	112	123	172	295	151	72
12 x 53	11.66	10.81	272	87.5	97	136	234	119	56
10 x 57	9.87	12.84	224	75.6	116	162	277	116	57
10 x 42	9.58	8.48	143	48.5	76	107	183	77	37

6.15.3.3 Buckling

The following shall replace the last sentence of A6.15.3.3.

The depth to fixity shall be determined in accordance with A10.7.3.13.4 for battered piles or COM624P or LPILE 5.0 analyses for vertical piles per D10.7.3.13.4. Figure D6.15.2-1P illustrates the depth to fixity as determined by COM624P or LPILE 5.0.

C6.15.3.3

The following shall replace AC 6.15.3.3.

The use of an approximate method in lieu of a P-Δ analysis is allowed if approved by the Chief Bridge Engineer.

SPECIFICATIONS

COMMENTARY

6.15.4 Maximum Permissible Driving Stresses

The following shall replace A6.15.4.

Maximum permissible driving stresses for top driven steel piles shall be taken as specified in D10.7.8.

6.16 REFERENCES

The following shall supplement A6.16.

Albrecht, P., Coburn, S.K., Wattar, F.M., Tinklenberg, G.L. and W.P. Gallagher. Guidelines for the Use of Weathering Steel in Bridges. NCHRP Report 314. TRB, National Research Council, Washington, D.C., June 1989.

Brockenbrough, R. L. 1970. "Criteria for Heat Curving Steel Beams and Girders," *Journal of the Structural Division*. American Society of Civil Engineers, Vol. 96, October 1970.

Brockenbrough, R. L. 1972. "Fabrication Aids for Continuously Heat Curved Girders," U.S. Steel Corporation, January 1972.

Brockenbrough, R. L. 1973. "Fabrication Aids for Girders Curved with V-Heats," U.S. Steel Corporation, January 1973.

Mertz, D. R., "Displacement-Induced Fatigue Cracking in Welded Steel Bridges", Ph.D. dissertation, Lehigh University, 1984

Modjeski and Masters, Inc., and Lehigh University. 2011 "Stress Distribution in Heat-Curved Girders and Resulting Limits on Heat Curving Radius," Final Report Submitted to Pennsylvania Department of Transportation, Bridge Quality Assurance Division, December 23, 2011.

SPECIFICATIONS

COMMENTARY

APPENDIX B6 – MOMENT REDISTRIBUTION FROM INTERIOR-PIER I-SECTIONS IN STRAIGHT CONTINUOUS-SPAN BRIDGES

Delete Appendix B in its entirety

The provisions of Appendix B correspond to the inelastic design procedures that are not allowed in Pennsylvania.

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7.4 MATERIALS

7.4.1 General

The following shall supplement A7.4.1.

The use of aluminum as bridge or other structural material is not allowed, except as depicted in Standard Drawings. If aluminum usage is prescribed by the Chief Bridge Engineer, applicable specifications will be determined or established at that time.

Aluminum appurtenances are allowable as per the BC Standard Drawings.

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8.4 MATERIALS

8.4.2 Metal Fasteners and Hardware

8.4.2.2 Minimum Requirements

8.4.2.2.2 Prestressing Bars

The following shall supplement A8.4.2.2.2.

For additional requirements on prestressing bars see A5.4.4.1 and D5.4.4.1.

8.11 BRACING REQUIREMENTS

8.11.2 Sawn Wood Beams

The following shall replace the third sentence of the first paragraph.

The spacing of intermediate bracing shall be based on lateral stability and load transfer requirements, but shall not exceed 20 ft..

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9.2 DEFINITIONS

The following shall supplement A9.2

Overhang Length - The length measured normal to the exterior girder to the edge to the deck.

The following shall replace the skew angle definition in A9.2.

Skew Angle - Angular measurement between the bridge centerline (or tangent thereto) and a line parallel to the support; a 90° skew angle defining a right bridge (See PP3.2.2).

9.4 GENERAL DESIGN REQUIREMENTS**9.4.3 Concrete Appurtenances**

The following shall replace A9.4.3.

Unless otherwise specified by the Chief Bridge Engineer, concrete curbs, barriers and dividers should be made structurally continuous.

Deflection joints in the concrete portion of the barrier shall be provided by a modified deflection joint detail as per Standard Drawing BC-752M. The spacing of the deflection joints shall be as per Standard Drawing BD-660M. Longitudinal barrier bars shall be continuous through the deflection joints.

9.5 LIMIT STATES**9.5.1 General**

The following shall replace the first paragraph of A9.5.1.

The structural contribution of a concrete appurtenance to the deck will be neglected for all limit states.

9.5.2 Service Limit States**9.5.5 Extreme Event Limit States**

The following shall replace A9.5.5.

Decks shall be designed for force effects transmitted by traffic, barriers and railings using loads, analysis procedures and limit states specified in A13 and D13.

For the barriers shown in the Standard Drawings, the deck and overhang designs given in Standard Drawing BD-601M are designed for the controlling condition.

C9.4.3

The following shall replace AC9.4.3.

Experience indicates that the interruption of concrete appurtenances at locations other than deck joints does not serve the intended purpose of stress relief. Large cracks, only 1 ft. or so away from open joints, have been observed in concrete barriers.

C9.5.1

Delete the first paragraph of AC9.5.1.

C9.5.2

The following shall supplement AC9.5.2.

Before any testing of the deck is begun (in order to determine the limits for excessive deck deformation), the Chief Bridge Engineer must review and approve the testing procedure. The results of this testing must also be approved by the Chief Bridge Engineer.

SPECIFICATIONS

COMMENTARY

9.6 ANALYSIS**9.6.1 Methods of Analysis**

The following shall replace A9.6.1.

Approximate elastic methods of analysis, specified in A4.6.2.1, shall be used for the various limit states in A9.5.

If approved by the Chief Bridge Engineer, refined methods, specified in A4.6.3.2, may be used for the various limit states in A9.5.

The empirical design of concrete slabs, specified in A9.7.2, will be used only if prescribed by the Chief Bridge Engineer.

9.7 CONCRETE DECK SLABS**9.7.1 General****9.7.1.1 Minimum Depth and Cover**

The following shall replace A9.7.1.1.

The minimum depth of concrete deck is 8 in. which includes a 1/2 in. wearing surface. Therefore, the minimum structural depth of concrete deck is 7 1/2 in..

Minimum cover shall be in accordance with the provisions of D5.12.3.

For composite adjacent box beam superstructures, the deck slab thickness shall be 5 1/2 in. minimum, including a 1/2 in. integral wearing surface with one mat of reinforcement. Slab thickness may need to be increased to provide minimum required cover for bridges made continuous for live load.

When AASHTO I-beams, Type V or VI, or PA Bulb-Tee beams are placed adjacent, the deck slab thickness shall be 7 1/2 in., including a 1/2 in. integral wearing surface with two mats of No. 4 reinforcement.

9.7.1.3 Skewed Decks

The following shall replace A9.7.1.3.

If the skew angle of the deck is from 90° to 75°, the primary reinforcement shall be placed parallel to the skew. If the skew angle of the deck is less than 75°, the primary reinforcement shall be placed perpendicular to the main supporting components. Standard Drawing BD-660M provides additional information.

9.7.1.5 Design of Cantilever Slabs**9.7.1.5.1P Overhang of Deck Slab on Concrete and Steel Girder Bridges**

The maximum overhang length shall not exceed either:

C9.7.1.1

Delete the last paragraph of AC9.7.1.1.

C9.7.1.3

The following shall replace AC9.7.1.3.

The intent of this provision is to prevent extensive cracking of the deck, which may be the result of having no appreciable reinforcement acting in the direction of principal flexural stresses due to a heavily skewed reinforcement, as shown in Figure AC9.7.1.3-1.

C9.7.1.5.1P

The overhang is measured from the centerline of the girder web for steel I beams. For P/S Box Beams the

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- the girder depth,
- requirements in A4.6.2.2, or
- requirements given in Table 9.7.1.5.1P-1 based on the exterior girder deflection.

For overhangs greater than 0.5S, the overhang and girder spacing must be approved by the Chief Bridge Engineer at the type, size and location stage.

Table 9.7.1.5.1P-1 – Exterior Girder Deflection Versus Overhand

Maximum LL+I Deflection Exterior Girder Δ	Maximum Overhang X
$L/800 = \Delta$	0.5 S
$L/1000 < \Delta < L/800$	Eq. 9.7.1.5.1P-1
$\Delta < L/1000$	0.625 S

$$X = S \left\{ 0.5 + 0.125 \left(\frac{0.00125L - \Delta}{0.00025L} \right) \right\} \quad (9.7.1.5.1P-1)$$

where:

X = maximum permitted overhang measured normal to the exterior girder from edge of deck to centerline of exterior girder (ft.)

S = Stringer spacing between the exterior girder centerline and adjacent interior girder centerline. When there is a variable stringer spacing, the stringer spacing is assumed to be the spacing between the exterior and interior girders at 1/3 of the span length measured from the narrow end of the stringer spacing (ft.)

L = span length (ft.)

Δ = maximum deflection of the exterior girder caused by design live load (excluding permit load), plus impact (ft.)

Any exception to the above criteria is permitted only with approval of the Chief Bridge Engineer.

The designer shall design and detail one method to stabilize the exterior girder to prevent excessive rotation and/or rollover of the exterior girder. In addition, a note shall be placed on the contract plans alerting contractors that the contractor shall check the need for the temporary

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overhang is measured from the centerline of the web.

Stabilizing the exterior girder of a steel bridge involves bracing the top and bottom flanges of the girder. The stabilization consists of a tension tie connecting the top flanges in conjunction with compression struts bracing the bottom flanges of the girders in the cross section. Typically

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bracing between the exterior girder and adjacent interior girder to prevent exterior girder rotation during the deck placement. The contractor will be required to submit for Department approval the drawings and calculations of the temporary bracing scheme.

The deck slab overhanging the girder adjacent to a longitudinal median joint shall have the same overhang and temporary bracing requirements as provided for the exterior girder.

9.7.1.6P Reinforcement Requirements

No transverse reinforcement bars larger than No. 6 shall be used in the deck slab.

Reinforcement in the compression face of the deck slab shall not be considered in the design.

Haunches greater than or equal to 3 in. shall be reinforced according to Standard Drawing BD-601M.

For deck protection guidelines, see D5.4.3.6P.

The minimum spacing of transverse bars shall not be less than 5 1/2 in..

9.7.1.7P Deck Construction Joints

Deck construction joints are permitted as shown on the plan. However, no material from any bulkhead type may remain in place.

9.7.2 Empirical Design

NOTE: An empirical design is only to be used if prescribed by the Chief Bridge Engineer.

9.7.2.2 Application**9.7.2.4 Design Conditions**

The following shall supplement A9.7.2.4.

The bulleted items in A9.7.2.4 are the minimum for an empirical design. For an empirical design to be valid in Pennsylvania, a deck must comply with PennDOT standards and these bulleted items.

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the bracing of the bottom flanges is constructed with timber struts and the restraint of the top flange is constructed with a structural shape such as an angle or even a reinforcing bar. The bracing and tension ties are typically placed at one-third points between cross frames. The flange tension tie can be embedded in the deck or placed below the deck as long as it does not interfere with the stay-in place deck forms. For top flanges in compression, a welded tension tie attachment to the top flange can be used. Tack welding a reinforcing bar to the stems of shear studs to provide a tension tie is not permitted. This note is required if the criteria in Table 9.7.1.5.1P-1 is violated and the Chief Bridge Engineer has given an exception.

C9.7.1.7P

Stayform sheets have also been used at bridge deck construction joints. This is a undesirable detail if the sheets are left in the deck, since it introduces a formed crack which would be a potential maintenance problem. In addition, corrosion will eventually develop in this joint, even though a galvanized metal sheet is provided. However, this detail may be used if the stayform sheet is removed after hardening of the concrete.

C9.7.2.2

Delete the second paragraph of AC9.7.2.2.

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9.7.2.5 Reinforcement Requirements

The following shall replace the last paragraph of A9.7.2.5.

If the skew is less than 75°, the specified reinforcement in both directions shall be doubled in the end zones of the deck. Each end zone shall be taken as a longitudinal distance equal to the effective length of the slab specified in A9.7.2.3.

9.7.4 Stay-in-Place Formwork**9.7.4.1 General**

The following shall replace the first bulleted item in the second paragraph of A9.7.4.1.

- 65% of the yield strength of steel, or

The following shall supplement A9.7.4.1.

The use of permanent metal deck forms shall be specified in all superstructure designs, unless it is not feasible. The use of removable forms should be called for on the plans as an alternative to the permanent metal deck forms.

Deck slab details and beam and diaphragm haunch details shall conform to details shown on the standards. The overall depth of forms and the support details shall not be shown on the design drawings.

There shall be no pay item for the forms.

If permitted by the Chief Bridge Engineer, prestressed concrete planks which form an integral part of the deck slab may be used in lieu of separate deck forms.

9.7.4.2 Steel Formwork

The following shall supplement A9.7.4.2.

Details of forms shall not be shown on the contract drawings, but applicable Standard Drawings for permanent metal deck forms shall be referred to on the supplemental drawings table on the contract drawings for details.

The maximum corrugation depth and width shall be such that the total dead load of the form and of the concrete in the form does not exceed 0.015 ksf and that the dead load deflection due to the weight of plastic concrete, deck steel reinforcement and form does not exceed the following:

- For design span lengths, S , less than or equal to 10 ft., $S/180$ or 1/2 in., whichever is less
- For design span lengths, S , greater than 10 ft., $S/240$ or 3/4 in., whichever is less.

The total of these loads, for design purposes, shall not be less than 0.180 ksf. The permissible form camber shall be based on the actual dead load condition. Camber shall

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C9.7.2.5

The following shall replace the last paragraph of AC9.7.2.5.

The intent of this provision is crack control. Beam slab bridges with a skew less than 65° have shown a tendency to develop torsional cracks due to differential deflections in the end zone, OHBDC (1983). The extent of cracking is usually limited to a width that approximates the effective length.

C9.7.4.1

The following shall supplement AC9.7.4.1.

It has been observed during construction that concrete placed by bucket or pumping operation may be 50% to 100% thicker than the design slab thickness at a particular location prior to vibrating and striking for finishing.

C9.7.4.2

Delete AC9.7.4.2.

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not be used to compensate for deflections in excess of the foregoing limits. Some new forms are manufactured in such a way as to eliminate concrete in the valleys.

In using permanent metal deck forms, an additional dead load consisting of the weight of the metal forms and the weight of the concrete in the valley of the forms shall be taken into account in the design.

The quantity of Class AAAP cement concrete shall be computed to include concrete in the valleys of the metal forms (this may be approximated by adding an extra 1 inch thickness to the deck), plus compensation for deflection. A note stating "Quantity of Class AAAP Cement Concrete includes Concrete in the Valleys of the Metal Forms" shall be shown under the table of "Summary of Quantities", where applicable.

9.7.4.3 Concrete Formwork*9.7.4.3.2 Reinforcement*

The following shall supplement A9.7.4.3.2.

For prestressed concrete planks, prestressing tendons shall protrude from the plank a sufficient distance to develop anchorage in the cast-in-place portion of the deck slab between the ends of the planks.

9.7.4.3.5P Additional Requirements

Precast prestressed deck planks used as permanent forms spanning between stringers shall be designed compositely with the cast-in-place portion of the slabs to support additional dead loads and live loads.

The planks shall be analyzed assuming they support their self-weight, any construction loads, and the weight of the cast-in-place concrete, and shall be analyzed assuming they act compositely with the cast-in-place concrete to support moments due to additional dead loads and live loads.

Live load moments shall be computed in accordance with A4.6.2.1.

In calculating stresses in the deck planks due to negative moment near the stringer, no compression due to prestressing shall be assumed to exist.

Deck planks shall be prestressed with pretensioned strands. The strands shall be in a direction transverse to the stringers when the planks are on the supporting stringers.

Reinforcing bars, or equivalent mesh, shall be placed in the plank transverse to the strands to provide at least 0.11 in^2 per foot of the plank.

C9.7.4.3.2

The following shall supplement AC9.7.4.3.2.

Before any testing of epoxy-coated strands is begun (in order to determine transfer and development lengths), the Chief Bridge Engineer must review and approve the testing procedure. The results of this testing must also be approved by the Chief Bridge Engineer.

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9.7.5 Precast Deck Slabs on Girders**9.7.5.1 General**

The following shall replace A9.7.5.1.

Both reinforced and prestressed precast concrete slab panels may be used. The minimum depth of the slab shall be as specified in D9.7.1.1.

All reinforcing steel and accessories (prestressed anchorage, couplers, etc.) shall be epoxy-coated for the full-depth deck panels in accordance with AASHTO M 284. Special provisions concerning precast deck units shall be furnished by the designer, including the deck unit fastening systems, prestressing system corrosion protection, post-tensioning requirements at the deck panel joints, fabrication and erection requirements, and detailed information on deck unit joints, deck unit adjustment, and, if required, deck unit beam composite interaction. Note that an unbonded post-tensioning system will not be permitted.

9.7.5.2 Transversely Joined Precast Decks

The following shall replace A9.7.5.2.

Transversely joined precast decks shall have sufficient bonded post-tensioning to cause the precast deck panels to behave as a continuous unit across the joints.

9.7.5.3 Longitudinally Post-Tensioned Precast Decks

The following shall replace the first paragraph of A9.7.5.3.

Longitudinally post-tensioned precast decks shall have sufficient bonded post-tensioning to cause the precast deck panels to behave as a continuous unit across the joints.

9.7.6 Deck Slabs in Segmental Construction**9.7.6.2 Joints in Decks**

The following shall replace A9.7.6.2.

Joints in the decks of precast segmental bridges may be epoxied match-cast surfaces, or cast-in-place concrete.

The strength of cast-in-place concrete joints shall not be less than that of the precast concrete. The width of the concrete joint shall permit the development of reinforcement in the joint or coupling of ducts, if used, but in no case shall it be less than 12 in..

Dry joints are not permitted in Pennsylvania.

9.8 METAL DECKS**9.8.1 General****C9.7.5.1P**

The requirements given in D9.7.5.2 and D9.7.5.3 for post-tensioning of the joints is to eliminate joint deterioration.

Precast deck may not be practical for:

- short spans or small bridges where the quantity of precast deck is very small,
- continuous bridges, or
- bridges on a curve.

C9.7.5.2

The following shall supplement AC9.7.5.2.

Shear key construction without bond post-tensioned is not allowed in Pennsylvania.

C9.8.1P

The use of steel beam flooring as currently manufactured, unless specifically designed for fatigue, should be avoided for projects whenever the anticipated

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total accumulative truck traffic will exceed 300,000 trucks, with a mass exceeding 6.0 kips per lane during the expected life of the flooring, unless the flooring is filled with concrete, either full-depth or half depth, as soon as possible after installation. Open steel beam bridge flooring has generally been a problem to maintain when subjected to heavy truck traffic, mainly as a result of the breaking of the welds and fatigue cracks (with ultimate fracture) of the cross bars, which are the most critical components of the grid floor. The flat-bar types appear to perform better relative to fatigue than the I-bar types.

Concrete-filled steel beam flooring performs satisfactorily relative to fatigue if properly connected to the stringers and girders. However, problems have been encountered with some filled steel grids growing in length and with welds breaking due to corrosion. Also, in some cases, apparently associated with the use of filled steel grid, a significant number of rivets and bolts have failed in stringer-to-floorbeam connections.

The first preference for a bridge deck should always be a conventional reinforced concrete slab.

9.8.2 Metal Grid Decks**9.8.2.1 General**

The following shall replace the third paragraph of A9.8.2.1.

Compute composite section properties for filled or partially filled decks as described in Standard Drawing BD-604M. Where filled or partially filled grid decks are modeled for analysis as either an orthotropic plate, or an equivalent grillage, flexural and torsional rigidities may be obtained by other accepted and verified approximate methods or by physical testing which must be approved by the Chief Bridge Engineer.

9.8.2.2 Open Grid Floors

Delete A9.8.2.2.

C9.8.2.2

Delete AC9.8.2.2

PennDOT does not allow open grid floors.

9.8.2.3 Filled and Partially Filled Grid Decks**9.8.2.3.1 General**

The following shall replace the third paragraph of A9.8.2.3.1.

A 1 1/2 in. thick structural overfill shall be provided.

The following shall supplement A9.8.2.3.1.

In the negative moment region, a welded connection of the grid deck to the beam or girder is not permitted.

Do not provide any deck joints, except as permitted at specific substructure locations permitted by the Chief Bridge Engineer.

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9.8.2.3.3 Fatigue and Fracture Limit State

The following shall supplement A9.8.2.3.3.

The welded internal connection among those elements of the steel grid which are not within the connection fill shall be considered as Category "E" details.

9.8.2.4 Unfilled Grid Decks Composite With Reinforced Concrete Slabs

Delete all the articles under A9.8.2.4.

9.8.3 Orthotropic Steel Decks**9.8.3.3 Wearing Surface**

The following shall supplement A9.8.3.3.

Before any testing of the orthotropic steel deck is begun (in order to determine long-term composite action between deck plate and wearing surface), the Chief Bridge Engineer must review and approve the testing procedure. The results of this testing must also be approved by the Chief Bridge Engineer.

9.8.5 Corrugated Metal Decks**C9.8.5P**

Delete all the articles under A9.8.5.

Pennsylvania does not permit this type of deck.

9.9 WOOD DECKS AND DECK SYSTEMS**9.9.3 Design Requirements****9.9.3.6 Skewed Decks**

The following shall replace the first sentence of A9.9.3.6.

Where the skew of the deck is greater than 65°, transverse laminations may be placed on the skew angle.

9.9.4 Glued Laminated Decks**9.9.4.1 General**

The following shall supplement A9.9.4.1.

The minimum nominal deck thickness for glued laminated decks is 6 in..

9.9.5 Stress Laminated Decks**9.9.5.1 General**

The following shall replace the second paragraph of A9.9.5.1.

Stress laminated decks shall not be used where the skew is less than 45°.

The following shall supplement A9.9.5.1.

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The deck thickness shall not be less than 8 in. nominal and the deck supports shall be continuous at abutments and piers.

9.9.5.6 Stressing*9.9.5.6.1 Prestressing System*

The following shall replace the second paragraph of A9.9.5.6.1.

In stress laminated decks, with skew angles greater than 65°, stressing bars may be parallel to the skew. For skew angles between 45° and 65°, the bars should be placed perpendicular to the laminations and, in the end zones, the transverse prestressing bars should either be fanned in plan as shown in Figure A9.9.5.6.1-1, or be in a stepped arrangement as shown in Figure A9.9.5.6.1-2.

The following shall supplement A9.9.5.6.1.

Type C prestressing configuration, given in Figure A9.9.5.6.1-3, is not permitted for soft wood laminations.

9.9.5.6.3 Design Requirements

The following shall supplement A9.9.5.6.3.

The shortest lamination must have at least two prestressing rods passing through. The spacing of the first rod from the lamination end should be generally equal to one-half the center-to-center spacing of prestressing rods and not less than the lamination thickness.

The camber for these decks shall be three times the dead load deflection.

9.9.5.6.4 Corrosion Protection

The following shall supplement A9.9.5.6.4.

Provide triple protection system for prestressing rods, i.e., plastic sleeves, grease and waterproof membrane.

9.11P DECK PROTECTION SYSTEMS**9.11.1P General**

This section outlines provisions for deck systems in the design and construction of new bridge decks for Federal, State and locally sponsored projects.

Polypropylene fibers or steel fibers may be specified in Class AAAP and HPC concrete decks to control cracking. If used, the contract must include special provisions.

A Single Deck Protection System is the minimum acceptable level of protection for all decks. Provide one deck protection system listed below:

- Filled and partially-fill metal grid deck.

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- Epoxy coated reinforcement.
- Galvanized reinforcement.

9.11.2P New Structures or Bridge Replacements

A Dual Deck Protection System or stainless steel reinforcement may be used for decks of new bridges on Federal Aid projects if approved by FHWA on a project-by-project basis.

The dual deck protection system must be a combination of deck protection systems as listed below:

- A. The use of epoxy-coated or galvanized reinforcement and the use of 1½ inches of latex modified or micro silica modified concrete overlay over an 8" minimum thickness Class AAAP or HPC concrete deck. Provide a 2 inch minimum clear cover over the top mat of reinforcement to the top of Class AAAP or HPC Cement Concrete instead of 2.5 inches of clear cover per PennDOT Standard Drawings BD-601M.
- B. Filled or partially-fill galvanized metal grid deck system, overfilled 1" during initial placement (overfilled monolithically) and the use of 1½ inches of latex modified or micro silica modified concrete overlay. Provide 1" overfill instead of 1½" per PennDOT Standard Drawings BD-604M.

The following may be used in lieu of a dual deck protection system:

- C. Stainless steel (solid stainless steel) top mat reinforcement and bottom mat reinforcement used with an 8" minimum thickness Class AAAP or HPC concrete deck. All ties, chairs and hardware in contact with reinforcement must be stainless steel.

9.11.3P Rehabilitation and/or Deck Replacements

Dual deck protection should only be considered on rehabilitation projects and/or deck replacement projects at the discretion of the District Bridge Engineer, based on the cost benefit of such protection and expected life of the remaining structure, and if approved by FHWA on a project-by-project basis (see D9.11.2P for dual protection system combinations).

If the expected life of the remaining structure is less than the expected life of a proposed new deck, a Single Deck Protection System is acceptable (see D9.11.1P for allowable single deck protection systems).

9.12P REFERENCES

See A9.10 for references.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 10 – FOUNDATIONS

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10.2 DEFINITIONS

The following shall supplement A10.2.

Competent Rock - A rock mass with discontinuities that are open not wider than 0.13 inches.

Design Load (DL) – Anticipated maximum service load in the micropile.

Driveability—A measure of the ability of a hammer/pile system to economically provide an undamaged installed pile with the required capacity.

End-Bearing Pile—A pile transferring a majority of its load to the soil and/or rock through friction or adhesion forces along the lower 1/3 of the pile shaft and bearing on the pile tip. (Same as combination point bearing and friction pile.)

Freeze—A natural phenomenon resulting in a time-dependent increase in pile strength occurring after driving. Freeze results from an increase in soil shear strength due to the dissipation of excess porewater pressures which develop in some cohesive soils during pile driving.

Karst—A set of physical conditions, landforms and bedrock attributes that may be present in areas that are underlain by bedrock that is soluble in water.

Overburden – Non-lithified material, natural or placed, which normally requires cased drilling methods to provide an open borehole to underlying strata

Pile Load Test—A test to determine pile capacity by the application of a static load.

Predetermined Pile Tip Elevation—The tip elevation, determined by the Engineer, from the pile load tests or test piles. The predetermined tip elevation locates the intended bearing strata for the point bearing or end bearing piles, or to verify or adjust the estimated tip elevations indicated.

Redriving—The act of driving a previously driven pile at some time after initial installation was completed. Redriving (also known as "restriking") is usually performed to evaluate the occurrence of freeze or relaxation. Evaluation may be made by comparison of driving resistances between the end of initial driving and the beginning of redriving, and/or the comparison of capacity estimates from dynamic monitoring at those times.

Relaxation—The time-dependent decrease in pile capacity occurring after driving, which typically occurs in relatively fine-grained soils and soft shales, claystones, or siltstones. Relaxation results from a decrease in soil or rock shear strength due to the dissipation of negative porewater pressures which develop in some very stiff cohesive soils and rocks (e.g., shales and claystones) during driving.

Residual Movement – The non-elastic (non-recoverable) movement of a micropile measured during load testing

Test Load (TL) – The maximum load to which the micropile is subjected during testing

Test Load Pile—A pile in a pile load test, also referred to as a load test pile.

Test Pile—A pile driven to verify the pile hammer's capability and to determine driving characteristics prior to driving test load and bearing piles.

The following shall replace the definition of Point-Bearing Pile in A10.2.

Point-Bearing Pile—A pile transferring a major portion of its load to the soil and/or rock through bearing on the pile tip. Only a minor portion of the load is transferred by friction or adhesion forces along the pile shaft.

10.3 NOTATION

The following shall supplement A10.3.

C_o	=	Uniaxial compressive strength of intact rock (ksf) (D10.6.3.2.2)
D_s	=	socket diameter (ft.) (D10.8.1.3)
J	=	Bearing capacity correction factor for footings on rock with widely spaced discontinuities (dim) (D10.6.3.2.3bP)
k	=	Lateral earth pressure coefficient (dim) (DC10.7.3.8.6c)
ℓ	=	span between adjacent units (ft.) (D10.5.2.2)
$N_{cq}, N_{\gamma q}$	=	Modified bearing capacity factors for effects of footing on or adjacent to sloping ground (dim) (D10.6.3.1.2c)
N_{ms}	=	Coefficient factor to estimate q_{ult} for rock (dim) (D10.6.3.2.2)
n	=	Exponential factor relating B/L or L/B ratios for inclined loading (dim) (A10.6.3.1.2a)
Q_n	=	nominal resistance (kips) (D10.9.3.5.2) (D10.9.3.7) (D10.9.3.8)
Q_R	=	factored resistance (kips) (D10.9.3.5.2) (D10.9.3.7) (D10.9.3.8)
Q_s	=	nominal micropile bond resistance (kips) (D10.9.3.5.2) (D10.9.3.7)
Q_{ug}	=	nominal uplift resistance of a micropile group (kips) (D10.9.3.8)
q_{max}	=	Maximum magnitude of footing contact pressure (ksf) (D10.6.5)
q_{min}	=	Minimum magnitude of footing contact pressure (ksf) (D10.6.5)
R	=	Resultant of pressure on base of footing (kips) (D10.6.5)
RQD	=	Rock Quality Designation (dim) (D10.6.3.2.2)
z_w	=	Depth from footing base down to the highest anticipated groundwater level (ft.) (D10.6.3.1.2gP)
δ'	=	Differential settlement between adjacent footings (ft.) (D10.5.2.2)
γ_m	=	Moist unit weight of soil (kcf) (D10.6.3.1.2gP)
ϕ_m	=	Angle of friction of rock mass (degrees) (D10.6.3.2.3aP)
ϕ_s	=	Resistance factor for the bond capacity of a micropile (dim) (D10.5.5.2.5)
κ	=	shear strength ratio (c_2/c_1) for two layered cohesive soil system below footing (dim) (A10.6.3.1.2e)
π	=	3.14
σ'_v	=	the effective vertical stress at midpoint of soil layer under consideration (ksf) (A10.7.3.8.6d)

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10.4 SOIL AND ROCK PROPERTIES**10.4.2 Subsurface Exploration**

The following shall supplement A10.4.2.

Perform subsurface explorations in accordance with Policies and Procedures, Chapter 6; and Publication 293, Geotechnical Engineering Manual, Part 2.

C10.4.2

The following shall supplement AC10.4.2

Areas of karst geology present special challenges because of the unique weathering, erosion, drainage and subsidence features that may be found there. Karst refers to a set of physical conditions, landforms, and bedrock attributes that may be present in areas that are underlain by bedrock that is soluble in water. In Pennsylvania, karst conditions are associated with carbonate rocks such as limestone and dolomite. The PA Geologic Survey (DCNR, 2000) has mapped the distribution of limestone and dolomite in Pennsylvania. Characteristics of karst areas include irregular and pinnacled bedrock surfaces; soil overburden mixed with rock fragments; and open cavities, conduits or caverns within the bedrock.

Sinkhole development is often associated with karst areas. Overburden soils are typically residual material resulting from weathering of the underlying bedrock. This residual soil can migrate or be washed into openings in the rock. As the soil continues to migrate into the openings in the rock a very soft zone or void develops in the overburden soil. When insufficient material remains to support the overlying soils, the roof collapses and a sinkhole develops. Sinkhole development is often associated with the movement of water. Sinkholes can be naturally occurring due to the percolation of surface water from natural drainage patterns. Sinkhole development can also be triggered by changes in drainage patterns due to development, construction activity that removes a portion of the overburden leaving insufficient material to bridge underlying voids, or dewatering and the associated drop in groundwater level. *For further description of sinkholes, sinkhole remediation and sinkhole prevention see the Ground Subsidence Management Guidelines (issued via SOL 421-08-05).*

Prior to planning a subsurface investigation in karst areas a thorough review of published and unpublished information should be performed in accordance with PP6.2.1. Carefully review historic aerial photographs to identify sinkhole scars, closed depressions and possible zones of bedrock fracturing. Stereoscopic aerial photographs are more effective than individual photos in identifying these features. Review available mapping of karst features and sinkholes (Kochanov).

During the review of published and unpublished information and the visual site inspection (PP6.2.2) particular attention should be paid to the presence of karst landforms such as sinkholes, closed depressions, resurgent springs and bedrock outcrops. Areas that farmers avoid could be pinnacles or sinkholes. Changes in vegetation sometimes indicate sinkhole activity. Forested areas in fully farmed lands may also indicate areas of shallow rock. Old sinkholes may have been used as dumps, and can be found

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below debris piles. Any trends in karst features should be noted (e.g. sinkhole alignment). Information on past sinkhole activity may be gained from interviews with local residents, municipal officials or local contractors.

Publication 293 provides guidance regarding selection of number and depth of borings, drilling techniques, sampling methods, in-situ testing and geophysical testing. Test borings with Standard Penetration Test (SPT) sampling in soil and rock coring are recommended for any subsurface investigation in karst areas. It is important that the boring logs include observations of such conditions as soft or wet zones in the soil overburden, drill water return (or the lack of), reaction of rock core samples to dilute HCL solution, and voids or soil-filled seams in bedrock. It is desirable to obtain SPT samples of soil seams in bedrock. In order to do this it will be necessary to advance casing below the top of rock or to use NX drilling tools. NX core barrels are large enough to accommodate a 2-inch split-barrel sampler for SPT sampling.

Test borings with SPT sampling and rock coring are relatively expensive and time consuming and provide data at the boring location only. Because subsurface conditions in karst areas are highly variable supplemental techniques such as pneumatic-powered, track-mounted percussion drilling (air-track), electronic cone penetrometer and geophysical methods can be valuable in obtaining a more complete understanding of subsurface conditions at the site. In all cases it is desirable to obtain data with the supplementary methods close to SPT/rock core borings so the results can be correlated.

Air-track drilling has the advantages of mobility, speed of drilling and relative economy. It is effective at penetrating boulders and ledges. An experienced operator can qualitatively detect voids, zones of broken rock and the soil/rock interface. However, when drilling rock, it is difficult to distinguish between raveling decomposed rock zones and zones of residual soil or between zones of soft soil and open cavities. Air-track drilling can be problematic through thick overburden or in intensely weathered bedrock with numerous clay-filled cavities and steeply sloping rock surface.

Use of an electric cone penetrometer can provide data on soft soils or cavities within the soil overburden, geotechnical data, and inferred top of rock at a relatively reasonable cost. Disadvantages are the inability to penetrate cobbles, boulders or ledges.

Since karst areas are characterized by highly variable subsurface conditions, geophysical investigations can be helpful to supplement and refine intrusive subsurface investigation programs, such as SPT, CPT, etc. See A10.4.5 for a general discussion of geophysical tests and references for detailed guidelines.

In karst areas, physical properties of interest include top of rock profile, voids or soil-filled seams in bedrock, voids in the overburden soils and soft zones in the overburden

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soils. Several geophysical methods can be used to obtain information on these properties.

Table C10.4.2-1P presents a summary of geophysical methods that are used in karst areas. The methods presented are not all inclusive, but represent the most commonly used methods. Table C10.4.2-1P is specific to the application of geophysical methods to karst areas.

Table C10.4.2-1P - Geophysical Methods Used In Karst Areas

GEOPHYSICAL METHOD	INFORMATION OBTAINED	LIMITATIONS
Seismic Refraction	- Depth to bedrock - Depth to water table - Cave detection and mapping - Fault and fracture delineation	- Sensitive to acoustic noise and vibrations - Not effective if stiffness decreases with depth or if soft layer underlies stiff layer (including a frozen surface layer) - Works best when sharp stiffness discontinuity is present - Provides limited information below top of rock surface
Electrical Resistivity	- Depth to bedrock - Depth to water table - Dissimilar strata - Fault and fracture delineation - Clay seam detection and mapping	- Resolution decreases with increasing depth - Susceptible to interference from nearby metal pipes, cables, or fences. - Heavy surface vegetation complicates data collection.
Electromagnetics (EM)	- Delineate areas of shallow/deep bedrock - Voids in soil	- Qualitative estimate of depths; extra effort required to characterize depth of target - Resolution decreases with increasing depth - Susceptible to interference from nearby metal pipes, fences, vehicles, noise from power lines and atmospheric storms.
Ground Penetrating Radar (GPR)	- Depth to bedrock - Depth to water table - Void detection	- Not effective below the water table or in clay. - Depth of penetration about 33 feet - Susceptible to interference from metal reinforcement, guardrails, and power lines.
Gravity	- Voids in soil and rock - Fault and fracture delineation	- Susceptible to interference from vibrations - Results are non-unique (i.e. more than one subsurface condition can give the same result) - Primarily large scale reconnaissance tool. - For voids in rock another geophysical method may be needed to delineate the bedrock surface to aid in data interpretation. - Void detectability decreases rapidly with depth.
Multichannel Analysis of Surface Wave (MSAW)	- Depth to bedrock - Depth to water table - Cave detection and mapping - Clay seam detection and mapping - Fault and fracture delineation	- Method relatively new, therefore less industry experience with results

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In addition to identifying methods that address physical properties of interest several additional considerations should be addressed when selecting a geophysical method, or combination of methods for use (Anderson, et al., 2003).

- What methods provide the required spatial resolution and target definition.
- What methods will perform well under the physical site constraints?
- What methods are cost effective?
- What methods provide complementary data?
- What non-geophysical control is required to refine the interpretation of the acquired geophysical data?
- Is the overall program, including intrusive subsurface investigations, cost effective?

While published guidelines referenced in A10.4.5 are helpful, the geophysical industry continues to develop and evolve. During the planning of any geophysical program it is recommended to consult an experienced geophysicist. Geophysical testing data should always be correlated with information from direct methods of exploration, preferably SPT/rock core borings. In addition, existing subsurface data should be shared with the geophysicist so that appropriate methods are selected and accurate interpretations are rendered.

10.4.3 Laboratory Tests

The following shall supplement A10.4.3

Soil and rock properties for use in the service limit state and strength limit state evaluation of foundations shall be based on the results of the field and/or laboratory testing. For additional information on field and laboratory testing, see Publication 293.

10.4.6 Selection of Design Properties

The following shall supplement A10.4.6.

Soil and rock properties for use in the service limit state and strength limit state evaluation of foundations shall be based on the results of the field and/or laboratory testing methods described in Publication 293.

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10.4.6.1 General**10.4.6.2 Soil Strength***10.4.6.2.3 Drained Strength of Cohesive Soils***10.4.6.4 Rock Mass Strength**

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C10.4.6.1

The following shall supplement AC10.4.6.1.

Soil and rock properties used in the design of foundations must represent the soil and rock mass as it will behave when subjected to loading by the actual foundation (i.e., use of γ , c and ϕ_t for undrained loading, and γ' , c' and ϕ'_t for drained loading in equations for geotechnical resistance.). The depth of soil below a foundation which is within the zone of loading influence typically ranges from two to four times the minimum plan dimension of the foundation element. For soil or rock that is generally homogeneous within the zone of influence, properties associated with the intact material are appropriate. This condition, however, is not typical. Usually, soil or rock within the zone of influence varies highly in consistency and contains bedding planes or other depositional features, soft seams, joints, fractures, or other discontinuities which will govern the behavior of the ground mass under load. In such cases, it is the strength and deformation characteristics of these critical features which should be evaluated.

C10.4.6.2.3

The following shall supplement AC10.4.6.2.3.

Correlations for long-term effective strength parameters should not be used for bridges or retaining walls with an exposed height greater than 10 ft.

C10.4.6.4

The following shall supplement AC10.4.6.4.

The rating adjustment for joint orientations presented in Table A10.4.6.4-2 requires a consideration of the effect of the joint orientation relative to the applied loading direction relative to the proposed construction. For example, for slope stability, an unfavorable orientation would be in a direction parallel to the load direction or inclined and dipping towards the open face. For settlement, an unfavorable orientation of a gouge-filled joint would be at right angles to the load direction resulting in closure of discontinuities and settlement.

Use caution in using RMF values when applying Hoek and Brown Criteria to poor to very poor quality rock masses. The methods of Hoek (2000), Chapter 11 utilizing the Geological Strength Index (GSI) can be used for these conditions.

Disturbance from stress relief and method of excavation can also impact the shear strength of rock masses. See Hoek (2002) for a modification of the Hoek and Brown criteria for disturbance.

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10.4.7P Special Soil, Rock and Other Problem Conditions

Geologic and environmental conditions can influence the performance of foundations and may require special consideration during design. To the extent possible, the presence and influence of such conditions shall be evaluated as part of the subsurface exploration program. A representative, but not exclusive, listing of problem conditions requiring special consideration is presented in Table 10.4.7P-1 for general guidance.

Table 10.4.7P-1 - Problem Conditions Requiring Special Consideration

PROBLEM TYPE	DESCRIPTION	COMMENTS
Soil	Organic Soil; Highly Plastic Clay	Low strength and high compressibility
	Sensitive Clay	Potentially large strength loss upon large straining
	Micaceous Soil	Potentially high compressibility (often saprolitic)
	Expansive Clay/Silt; Expansive Slag	Potentially large expansion upon wetting
	Liquefiable Soil	Complete strength loss and high deformations due to earthquake loading
	Collapsible Soil	Potentially large deformations upon wetting (caliche; loess)
	Pyritic Soil	Potentially large expansion upon oxidation, potentially corrosive environment
Rock	Laminated Rock	Low strength when loaded parallel to bedding
	Expansive Shale	Potentially large expansion upon wetting; degrades readily upon exposure to air/water
	Pyritic Shale	Expands upon exposure to air/water, degrades readily upon exposure, potentially corrosive environment
	Soluble Rock	Soluble in flowing and standing water (limestone, dolostone, gypsum)
	Weak Claystone (Red Beds)	Low strength and readily degradable upon exposure to air/water
	Gneissic and Schistose Rock	Highly distorted with irregular weathering profiles and steep discontinuities
Condition	Subsidence	Typical in areas of underground mining or high groundwater extraction
	Sinkholes/Solutioning	Karst topography; typical of areas underlain by carbonate rock strata
	Negative Skin Friction/Expansion Loading	Additional compressive/uplift load on deep foundations due to settlement/uplift of soil
	Corrosive Environments	Acid mine drainage; degradation of certain soil/rock types
	Permafrost/Frost	Typical in northern climates
	Capillary Water	Rise of water level in silts and fine sands leading to strength loss

See Publication 293M for conditions typical of the Commonwealth of Pennsylvania for preliminary identification of the potential for special problem conditions at a site.

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10.4.7.1P Problem Soils*10.4.7.1.1P Organic Soils*

Organic soils, such as peats and organic silts and clays, are common to most lacustrine, estuarine, and fluvial environments. These soils exhibit low strength and excessive deformability. Excessive deformability can result in large settlements which may place additional downward forces (i.e., negative skin friction) on piles and drilled shafts.

10.4.7.1.2P Sensitive Clays

Sensitive clays exhibit significant loss of strength due to disturbance. Poor construction practice in sensitive clays can result in additional downward forces (i.e., negative skin friction) on piles and drilled shafts due to excessive settlement of the surrounding soil relative to the pile or shaft, and in reduced axial and lateral load capacity due to soil strength loss.

10.4.7.1.3P Micaceous Soils

Micaceous soils result from the in-place weathering of igneous and metamorphic rocks. The presence of mica in the soil matrix is indicative of lower strength and higher compressibility than similar soils without mica. If micaceous soils are present, laboratory and/or field testing shall be performed to evaluate the soil strength and compressibility.

10.4.7.2P Problem Rocks*10.4.7.2.1P Shales*

Certain shales exhibit little frictional resistance along natural bedding planes and can be unstable if loaded other than normal to the bedding surfaces. Some shales expand markedly when unloaded and exposed to the air due to absorption of water by the clay minerals. Other shale types, pyritic shales for instance, expand during chemical weathering of the rock. In addition, sulfuric acid formed during the oxidation of the pyrite deteriorates concrete because of its deleterious effect on cement. When these conditions are anticipated or encountered, provisions shall be taken to minimize their impact on foundation performance, including extending foundations to greater depths, minimizing the time the foundation excavation remains open during construction, and using sulfate resistance cement.

10.4.7.2.2P Soluble Rocks

Open channels and joints, caverns, sinkholes,

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discontinuities and irregular top of rock topography are characteristic of soluble carbonate rock types (e.g., limestone and dolostone) where special attention shall be given to the potential for solution features. When soluble rock formations are encountered, consideration shall be given to extending drilled shafts or micropiles below the soluble formations and to the potential for additional lateral and downward loading on the shaft or micropile due to collapsed rock. Publication 408 requires probing a minimum depth of ten times the shaft diameter below the tip of each drilled shaft prior to concreting.

10.4.7.2.3P Gneissic and Schistose Rocks

Gneissic and schistose rock formations are the consequence of metamorphic geologic processes that result in banded and distorted strata and irregular weathering profiles. In designing drilled shafts in these rock types, consideration shall be given to defining the top of rock, the depth of weathering and the potential for instability where the tip of the shaft is installed in steeply dipping strata.

10.4.7.2.4P Claystone

Piles bearing on claystone have the potential to experience long-term settlement. The District Bridge Engineer and/or District Geotechnical Engineer will make the determination for estimated pile tip elevation and bearing material due to the significant remedial effort required for piles bearing on claystone subject to long term settlement.

If claystone material is present at or near the estimated pile tip elevation, consideration should be given to driving through the claystone layer for piles to bear on competent rock. Predrilling of piles is recommended for penetrating claystone layers 2 ft. or more in thickness to ensure that the piles obtain refusal at the estimated pile tip elevation in the specified bearing material.

Foundations should not be terminated in claystone unless a slake test has been performed and indicates that the claystone is not subject to deterioration.

10.4.7.3P Other Problem Conditions*10.4.7.3.1P Mine Voids*

Subsidence or caving of mine voids can lead to extreme and unpredictable settlements. Total collapse of a foundation may occur if an underlying mine void goes undetected during the site subsurface exploration. Slumping soil about a drilled shaft can cause additional loading on the shaft similar to the effects of negative skin friction where a

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C10.4.7.2.4P

Past experience in Allegheny County has shown long term settlement of bearing piles founded in claystone material.

Claystone is a fine-grained, dark gray, greenish-gray, pale green, pink, or purple sedimentary rock consisting primarily of compacted and hardened clay. Claystone is similar to shale but without laminations. Claystone can be susceptible to degradation when exposed to air or water, and this degradation may be initiated or aggravated by the installation of piles, caissons, or other deep foundations. When a claystone layer overlies the proposed bearing stratum, it is possible for piles to obtain refusal in the claystone layer. Note, a concern with deep foundations terminated in Claystone is the potential for long-term settlement.

Where claystone material may be present, sufficient core borings or exploratory holes will be helpful in determining the extent of predrilling required to ensure that the piles are being driven through the claystone strata and reaching refusal on competent rock.

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shaft is installed through compressible soils.

10.4.7.3.2P Corrosive Environments

The oxidation of pyrite in pyritic shales can create a corrosive environment for concrete by formation of sulfuric acid. Other sources of sulfate species are soils, groundwater, coal and acid mine drainage or industrial runoff. Use of sulfate-resistant cement in the concrete or a high quality watertight concrete shall be considered when deterioration from sulfate is a potential problem.

Large concentrations of chlorides are sometimes present in groundwater, soils and industrial runoff, and may contribute to the corrosion of steel. When sulfur species are present in combination with chlorides, the corrosive process is accelerated. When these conditions are encountered, consideration shall be given to protecting the reinforcing steel with additional concrete cover, using a greater cross-sectional area of reinforcement to compensate for long-term loss, or using high-strength reinforcing which provides greater corrosion resistance than mild steel.

10.5 LIMIT STATES AND RESISTANCE FACTORS**10.5.1 General**

The following shall supplement A10.5.1.

Foundations shall be designed to support the appropriate load combinations without structural failure of the foundation or a bearing resistance failure of the supporting soil and/or rock (strength limit state), or unacceptable settlements and associated structural deformations (service limit state). The foundation load combinations and tolerable structural deformations are controlled by the superstructure and substructure design. The resistances and movements of the selected foundation type shall be estimated using the procedures prescribed in D10.6 (Spread Footings), D10.7 (Driven Piles), D10.8 (Drilled Shafts), and D10.9 (Micropiles).

10.5.2 Service Limit States**10.5.2.1 General****C10.5.2.1**

The following shall supplement AC10.5.2.1.

Estimated allowable bearing pressures were developed for use with working stress design for shallow foundations on rock. These values may be used for preliminary sizing of foundations. See Appendix DB10P.

10.5.2.2 Tolerable Movements And Movement Criteria**C10.5.2.2**

The following shall supplement A10.5.2.2.

Allowable settlement criteria for footings on soil shall be developed by the structural designer consistent with the

The following shall supplement AC10.5.2.2.

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function and type of structure, the anticipated service life and the consequences of unacceptable settlements on the performance of the structure. Foundation settlement analyses shall be based on the results of in situ and/or laboratory testing to characterize the load deformation behavior of the foundation soils. Settlement analyses shall be performed to determine the relationship between estimated settlement and footing bearing pressure to optimize the footing size with respect to the loads to be supported.

The allowable settlement for shallow footings supporting bridge structures shall be based on the angular distortion (δ'/ℓ) between adjacent support units (i.e., between piers or piers and abutments) where δ' and ℓ are the differential settlement and span between adjacent units, respectively. In addition, the maximum net settlement of a footing shall not exceed 1 inch. The dimensionless ratio δ'/ℓ shall be limited to 0.0025 and 0.0015 for simple and continuous span bridges, respectively. Special treatment shall be given to differential settlement in a rigid frame type of structure (e.g., pier bent, rigid frame bridge, etc.). Rigid frames shall be designed for anticipated settlements. The above allowable differential settlement limits do not apply to rigid frame structures.

10.5.3 Strength Limit States**10.5.3.1 General****10.5.5 Resistance Factors****10.5.5.2 Strength Limit States***10.5.5.2.2 Spread Footings*

The following shall replace A10.5.5.2.2.

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of site specific considerations in A10.5.5.2.1.

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Differential settlement between substructure units results in stress redistribution in continuous beams. Large total settlements reduce vertical clearance and result in misalignment of approach roadway.

The net settlement of a footing is the settlement that occurs after the supported columns or beams are set and framed.

C10.5.3.1

The following shall supplement AC10.5.3.1.

Scour investigations shall be made in accordance with PP7.2.

C10.5.5.2.2

The following shall supplement AC10.5.5.2.2.

Regionally specific values have also been incorporated in to bearing resistance factors for theoretical methods in sand.

The resistance factors for sliding of shallow footings on sand are higher for precast footings than for cast-in-place footings, indicating that estimation of sliding resistance is more reliable for precast footings. However, as indicated by Equation A10.6.3.4-2, the nominal resistance of a precast footing is only 80% of the nominal resistance of a cast-in-place footing, primarily because the precast footing has a formed base.

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Table 10.5.5.2.2-1 - Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State.

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Bearing Resistance	ϕ_b	Sand	
		- Semi-empirical procedure using SPT data	0.45
		- Semi-empirical procedure using CPT data	0.45
		- Theoretical Estimation	
		using ϕ_t estimated from SPT data	0.45
		using ϕ_t estimated from CPT data	0.50
		using ϕ_t measured directly in lab or field tests	0.50
Sliding	ϕ_τ	Clay	
		- Semi-empirical procedure using CPT data	0.45
		- Theoretical Estimation	0.50
		using shear resistance measured in lab tests, using shear resistance measured in field vane tests, using shear resistance estimated from CPT data	
		Rock	
		- Semi-empirical procedure, Carter and Kulhawy (1988)	0.50
		Plate Load Test	0.55
	ϕ_τ	Precast concrete placed on sand	
		using ϕ_t estimated from SPT data	0.90
		using ϕ_t estimated from CPT data	0.90
		using ϕ_t measured directly in lab or field tests	0.90
		Concrete cast-in-place on sand	
		using ϕ_t estimated from SPT data	0.80
		using ϕ_t estimated from CPT data	0.80
		using ϕ_t measured directly in lab or field tests	0.80
	ϕ_τ	Precast concrete placed on rock	
		using δ from Table A3.11.5.3-1	1.00
		using δ measured directly in lab or field tests	0.90
		Concrete cast-in-place on rock	
		using δ from Table A3.11.5.3-1	1.00
		using δ measured directly in lab or field tests	0.80

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COMMENTARY

Table 10.5.5.2.2-1 - Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State (Continued)

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Sliding	ϕ_τ	Precast or cast-in-place concrete on clay	0.85
		Soil on soil	1.0
	ϕ_p	Passive earth pressure component of sliding resistance	0.50

10.5.5.2.3 Driven Piles

The following shall replace Table 10.5.5.2.3-1 in A10.5.5.2.3.

Table 10.5.5.2.3-1 - Resistance Factors for Driven Piles.

Condition/Resistance Determination Method		Resistance Factor
Nominal Resistance of Single Pile in Axial Compression—Dynamic Analysis and Static Load Test Methods, ϕ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.8
	Driving criteria established by dynamic testing*, quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
Nominal Resistance of Single Pile in Axial Compression—Static Analysis Methods, ϕ_{stat}	Side Resistance and End Bearing: Clay and Mixed Soils α -method (Tomlinson, 1987; Skempton, 1951)	0.70
	β -method (Esrig & Kirby, 1979; Skempton, 1951)	0.50
	λ -method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.55
	Side Resistance and End Bearing: Sand Nordlund/Thurman Method (Hannigan et al., 2005)	0.50
	SPT-method (Meyerhof)	0.45
	CPT-method (Schmertmann)	0.55
Block Failure, ϕ_{bl}	Clay	0.65

* Dynamic testing requires signal matching, and best estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to the static load test, when available.

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Table 10.5.5.2.3-1 - Resistance Factors for Driven Piles. (Continued)

Uplift Resistance of Single Piles, ϕ_{up}	Nordlund Method	0.40
	α -method	0.60
	β -method	0.40
	λ -method	0.45
	SPT-method	0.35
	CPT-method	0.45
	Load test	0.80
Group Uplift Resistance, ϕ_{ug}	Sand and clay	0.55
Lateral Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles	See the provisions of D6.5.4.2
	Concrete piles	See the provisions of D5.5.4.2.1
	Timber piles	See the provisions of A8.5.2.2 and A8.5.2.3
Pile Drivability Analysis, ϕ_{da}	Steel piles	See the provisions of A6.5.4.2
	Concrete piles	See the provisions of A5.5.4.2.1
	Timber piles	See the provisions of A8.5.2.2
	In all three Articles identified above, use ϕ identified as “resistance during pile driving”	

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10.5.5.2.4 Drilled Shafts

The following shall replace Table 10.5.5.2.4-1 in A10.5.5.2.4.

Table 10.5.5.2.4-1 - Resistance Factors for Geotechnical Resistance of Drilled Shafts.

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, ϕ_{stat}	Side resistance in clay	α -method (O'Neill and Reese, 1999)	0.65
	Tip resistance in clay	Total Stress (O'Neill and Reese, 1999)	0.55
	Side resistance in sand	β -method (O'Neill and Reese, 1999)	0.65
	Tip resistance in sand	O'Neill and Reese (1999)	0.55
	Side resistance in IGMs	O'Neill and Reese (1999)	0.65
	Tip resistance in IGMs	O'Neill and Reese (1999)	0.55
	Side resistance in rock	Horvath and Kenney (1979) O'Neill and Reese (1999) Carter and Kulhawy (1988)	0.65 0.55 0.55
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) O'Neill and Reese (1999)	0.50 0.50 0.50
Block Failure, ϕ_{bf}	Clay		0.65
Uplift Resistance of Single-Drilled Shafts, ϕ_{up}	Clay	α -method (O'Neill and Reese, 1999)	0.55
	Sand	β -method (O'Neill and Reese, 1999)	0.55
	Rock	Horvath and Kenney (1979) Carter and Kulhawy (1988)	0.55 0.45
Group Uplift Resistance, ϕ_{ug}	Sand and clay		0.55
Lateral Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), ϕ_{load}	All Materials		0.70
Static Load Test (uplift), ϕ_{upload}	All Materials		0.60

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10.5.5.2.5 Micropiles

The following shall replace A10.5.5.2.5.

Resistance factors for micropile foundation systems at the strength limit state shall be taken as specified in Table 10.5.5.2.5-1. Resistance factors for the service limit state shall be taken as 1.0.

Table 10.5.5.2.5-1 – Resistance Factors for Geotechnical Resistance of Axially Loaded Micropiles

Limit State	Method / Ground Condition	Resistance Factor
Axial Compressive Resistance of Single Micropile, ϕ_s	Side (Shaft) Resistance: Presumptive Values (1) Load Test (2)	Rock = 0.60 Rock = 0.80
Uplift Resistance of Single Micropile, ϕ_{up}	Side (Shaft) Resistance: Presumptive Values (1) Load Test (2)	Rock = 0.60 Rock = 0.80
Group Uplift Resistance, ϕ_{ug}	Side (Shaft) Resistance: Presumptive Values (1) Load Test (2)	Rock = 0.60 Rock = 0.80

- (1) Apply to Presumptive grout-to-ground bond values for preliminary design only in DC10.9.3.5.2
- (2) Apply where preproduction load tests are conducted to a load of 1.0 or greater times the factored design load on individual micropiles

Table 10.5.5.2.5-2 – Resistance Factors for Structural Resistance of Axially Loaded Micropiles

Section / Loading Condition		Resistance Factor
Pile Cased Length	Tension, ϕ_{TC}	0.80
	Compression, ϕ_{CC}	0.65
Pile Uncased Length	Tension, ϕ_{TU}	0.80
	Compression, ϕ_{CU}	0.65

10.5.5.3 Extreme Limit States

10.5.5.3.2 Scour

The following shall replace A10.5.5.3.2.

The foundation shall be designed so that the resistance (stability) remaining after the maximum scour, resulting from the scour design flood or the superflood, as defined in PP7.2.2 or PP7.2.3, respectively, has occurred, provides adequate foundation resistance to support the Service I limit state loadings.

The foundation shall resist not only the loads applied from the structure but also any debris loads occurring during the flood event.

C10.5.5.3.2

The following shall replace AC10.5.5.3.2.

Stability of deep foundations, including unsupported length of piles, must be analyzed with the maximum scour depth at the Service I limit state, as specified in D3.7, to ensure the bridge can remain operational until the extent of damage can be evaluated and repaired.

For the analysis of foundations with water levels exceeding the scour design flood, the extreme event II limit state, utilizing corresponding water level, water loads, and scour depth, may be used.

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10.6 SPREAD FOOTINGS**10.6.1 General Considerations****10.6.1.2 Bearing Depth**

The following shall supplement A10.6.1.2.

Where the potential for scour, erosion or undermining exists, spread footings shall be located to bear below the maximum anticipated depth of scour, erosion, or undermining as specified in A2.6.4.4, and PP7.2.

For footings constructed on slopes, a minimum horizontal bench 4 ft. wide shall be provided as shown in Figure D11.1.1.1.1P-1. The actual bench width shall consider overall stability of the slope in accordance with D10.6.2.5. In no case shall the minimum embedment depth to the bottom of footings be less than 3 ft. or the expected depth of frost penetration, nor shall the soil cover over the footing be less than 1 ft..

Footing depths and scour protection must follow the provisions of PP7.2 when scour is a possibility. Scour investigations shall be made in accordance with PP7.2.

10.6.1.4 Bearing Stress Distributions

The following shall supplement A10.6.1.4.

The value of q_r obtained using the reduced footing dimensions represents an equivalent uniform bearing pressure and not the actual contact pressure distribution beneath the footing. This equivalent uniform bearing resistance shall be compared to the factored bearing pressure in accordance with D11.6.3.2. The actual contact pressure distribution shall be used for structural design of the footing as described in D10.6.5.

10.6.1.5 Anchorage Of Inclined Footings

The following shall replace A10.6.1.5.

Inclined footing bases shall not be used without the approval of the Chief Bridge Engineer.

C10.6.1.4P

Eccentric loads have the effect of reducing the bearing resistance of a footing. "Actual" footing contact pressures for eccentrically loaded footings are typically calculated assuming a rigid footing.

For non-rectangular sections, L may be estimated as follows:

Abutment L = Bearing-to-bearing + 4 ft.

Wing L = Construction Joint to End of Wing

C10.6.1.5

Delete AC10.6.1.5

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10.6.1.9P Replacement Of Unsuitable Material

Where unsuitable material exists to a depth of 10 ft. or less below the footing bearing level, consideration shall be given to the use of spread footings on compacted structure backfill as an alternative to deep foundations. Where unsuitable material extends more than 10 ft. below the footing bearing level, deep foundations shall be used. Unsuitable material is defined as material which will not provide adequate bearing capacity with acceptable settlements as determined in accordance with sections A10.6.2, D10.6.2, A10.6.3, and D10.6.3.

To permit the use of footings, unsuitable material shall be removed and replaced as shown in Figure 10.6.1.9P-1. For footings on clay soil which provide adequate bearing resistance, a minimum overexcavation of 6 in. shall be required. All unsuitable material removed below the footing bearing level shall be replaced with compacted structure backfill, and all unsuitable material above the footing bearing level shall be replaced with compacted embankment material in accordance with the Standard Drawings (see Standard Drawing RC-12M).

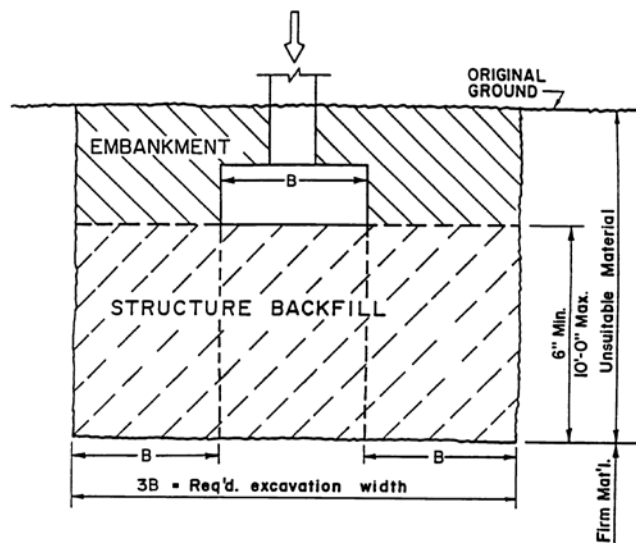


Figure 10.6.1.9P-1 - Unsuitable Material Replacement

10.6.2 Service Limit State Design**10.6.2.4 Settlement Analyses***10.6.2.4.1 General*

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C10.6.1.9P

OSHA regulations in regard to excavation must be followed in all cases of removal and replacement of unsuitable material.

Instances which require a relatively shallow depth of removal and replacement of unsuitable material may not require excavation width to be as wide as $3B$, as shown in Figure 10.6.1.9P-1. A narrower excavation width will be acceptable provided engineering analysis shows that the factored bearing resistance will be adequate and the foundation settlement will be acceptable.

C10.6.2.4.1

The following shall supplement AC10.6.2.4.1.

Methods for determining the distribution of vertical stress below a loaded area are usually based on elastic theory. While most soils are not elastic, the approach is

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valid, provided the additional stress imposed by foundation loading is below about 75% of the failure stress.

For highway applications, the stress distributions of most value include those presented in Figures C10.6.2.4.1-1P and C10.6.2.4.1-2P. The charts presented in Figure C10.6.2.4.1-1P present solutions for the cases of (a) the vertical stress increase below the corner of a uniformly loaded rectangular area and (b) the vertical stress increase at various locations below a uniformly loaded circular area. By applying superposition principles, the solution for the vertical stress below the corner of a uniformly loaded rectangular area may be used to determine the vertical stress increase below the center or point along the edge of a rectangular or square area. The charts presented in Figure C10.6.2.4.1-2P provide solutions for the distribution of vertical stress increase below areas having a variable intensity of surface loading typical of an embankment.

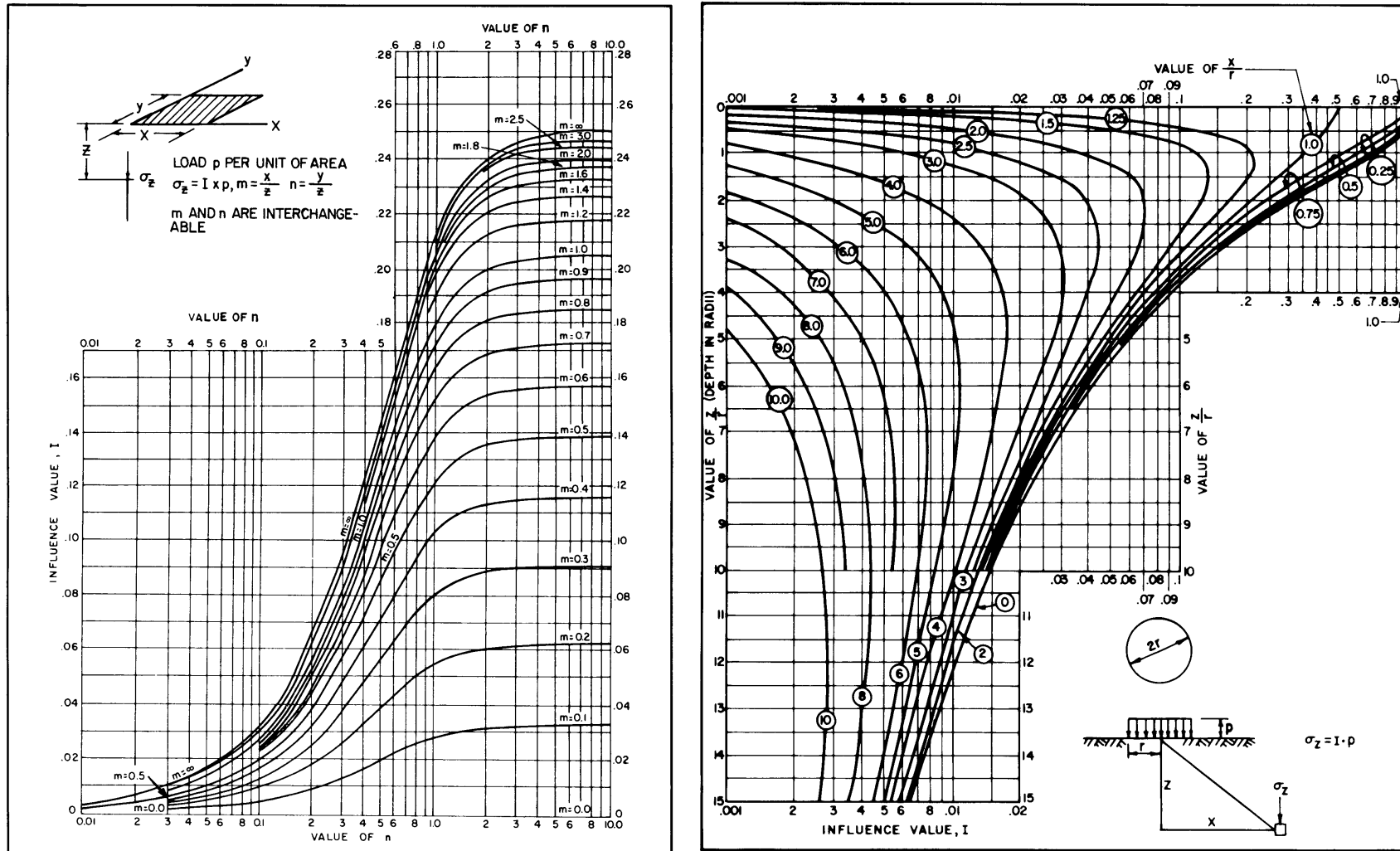


Figure C10.6.2.4.1-1P - Influence Coefficients for Vertical Stress Below Uniformly Loaded Rectangular and Circular Areas (U. S. Department of the Navy, 1986)

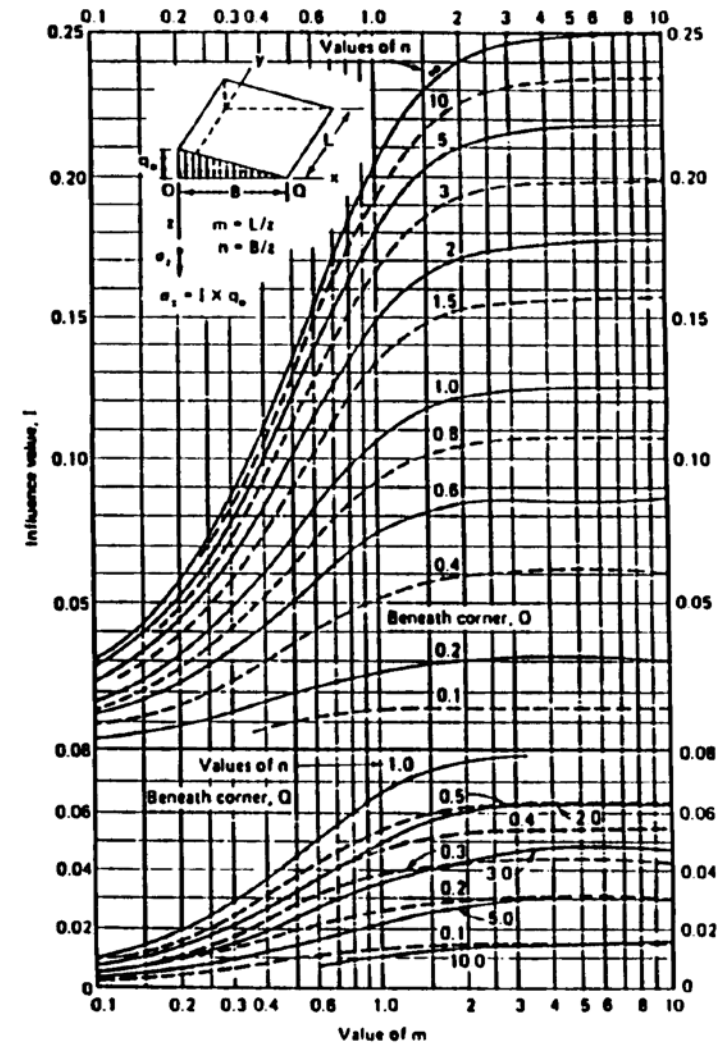
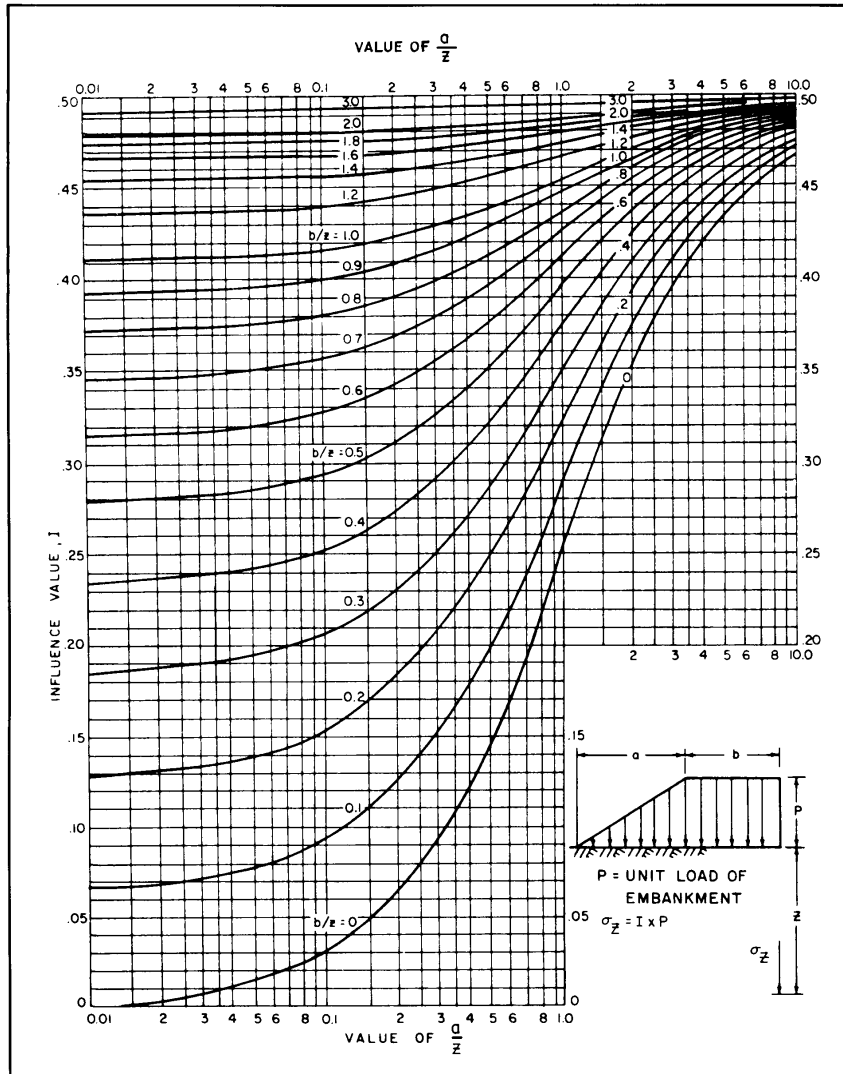


Figure C10.6.2.4.1-2P - Influence Coefficients for Vertical Stress Below Embankment and Triangular Areas (U. S. Department of the Navy, 1986)

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10.6.2.4.2 Settlement of Footings on Cohesionless Soils

The following shall replace the third paragraph of A10.6.2.4.2.

The average elastic settlement of footings on cohesionless soils and stiff cohesive soils may be estimated using the following:

$$S_e = \mu_0 \mu_1 \frac{q_o B'}{144 E_s} \quad (10.6.2.4.2-1)$$

where:

- μ_0 = depth factor taken as 1.0
- μ_1 = layer thickness coefficient taken from Figure 10.6.2.4.2-2P
- q_o = vertical stress at base of loaded area (ksf)
- B' = effective footing width as specified in A10.6.1.3 (ft.)
- E_s = Young's modulus of soil taken as specified in AC10.4.6.3 if direct measurements of E_s are not available from the results of in situ or laboratory tests (ksi)

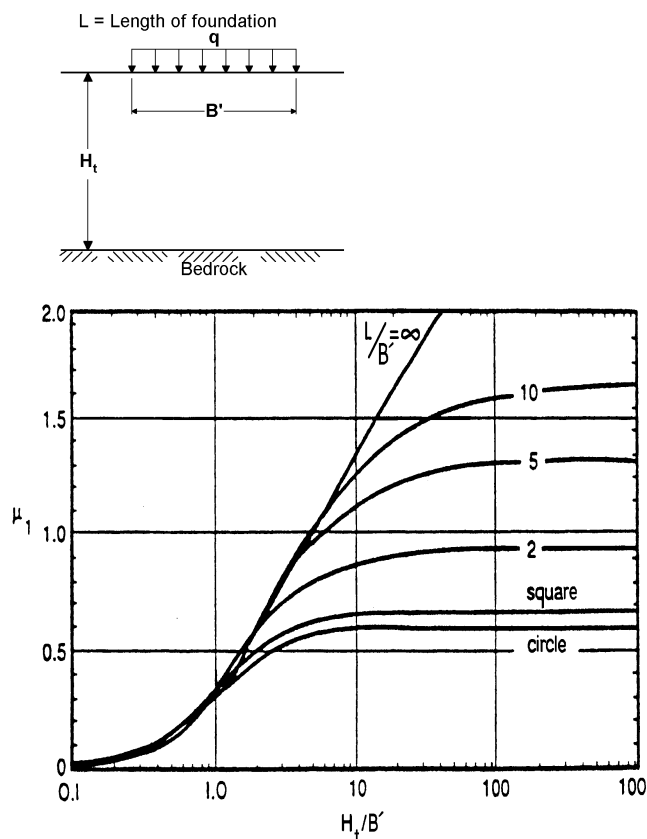


Figure 10.6.2.4.2-2P - Settlement Influence Factor μ_1 after Christian and Carrier (1978)

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C10.6.2.4.2

The following shall supplement AC10.6.2.4.2.

Elastic settlement is the principal component of the total settlement experienced by footings on granular soils and stiff cohesive soils.

When more than one layer of soil is encountered beneath a footing, the elastic settlement of each layer shall be calculated separately using the layer thickness coefficient for that layer. To determine the effective width B' for use in Figure 10.6.2.4.2-2P, the stress, q_n , at the top of a given layer is assumed to be distributed over an effective area which has been increased by the depth to the layer, H , resulting in an effective width of $B' + H$.

When calculating the settlement for piers on multi-layered soils, the length, L , should also be increased proportionately. The effective footing length L would be $L + H$ and the effective footing width would be $B' + H$, when selecting the appropriate curve for the soil layer under consideration. The stress used in Equation 10.6.2.4.2-1 would be calculated as per Equation C10.6.2.4.2-1P.

$$q_n = q_o [B' / (B' + H)] [L / (L + H)] \quad (C10.6.2.4.2-1P)$$

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10.6.2.4.3 *Settlement of Footings on Cohesive Soils*

The following shall supplement A10.6.2.4.3.

For foundations on stiff cohesive soils, the elastic settlement may be determined using Equation D10.6.2.4.2-1.

10.6.2.4.4 *Settlement of Footings on Rock*

The following shall supplement A6.10.2.4.4.

If soil-filled discontinuities are present below a footing, settlement of such layers should be estimated using the procedures for footings on soil.

10.6.2.5 Overall Stability

The following shall replace A10.6.2.5.

The overall stability of footings, slope and foundation soil or rock shall be evaluated using Load Combination Service I Limit State; the provisions of A3.4.1, D3.4.1, D3.4.1.1P, A10.5.2.3 and D11.6.2.3 and the resistance factors given in Table 10.6.2.5-1P.

Table 10.6.2.5-1P - Minimum Resistance Factors, ϕ , for Overall Stability at the Service Limit State

GEOMETRY	SPT* AND VISUAL CLASSIFICATION	SPT* AND LAB CLASSIFICATION	SPT* AND LAB STRENGTH TESTING
General Spread Footing	0.55	0.65	0.75
Abutment Supported Above Retaining Wall	Not Acceptable	0.55	0.65

* Standard Penetration Test data from soil borings.

The overall stability shall be evaluated using limiting equilibrium methods of analysis, such as those employed by the GSTABL computer program, (Gregory Geotechnical Software, 2003). Depending on whether the anticipated

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C10.6.2.4.3

The following shall replace the third paragraph of AC10.6.2.4.3.

To account for the decreasing stress with increased depth below a footing and variations in soil compressibility with depth, the compressible layer should be divided into vertical increments, i.e., typically 5.0 to 10.0 ft. for most normal width footings for highway applications, and the consolidation settlement of each increment analyzed separately. The number and thickness of compressible layers will depend on the total depth of compressible soil below the footing and the desired accuracy of the settlement computations. In general, layers should be thinnest near the footing base to account for rapid changes in vertical stress distribution due to the applied stress, and variations in σ_p' with depth. The total value of S_c is the summation of S_c for each increment.

The following shall supplement AC10.6.2.4.3.

The use of spread footings when total consolidation settlement exceeds serviceability limitations is undesirable and another foundation type should be considered.

C10.6.2.5P

Overall stability refers to deep seated failures.

Until a computer program is developed to analyze overall stability which incorporates the LRFD methodology, the Department will use the factor of safety method where $S.F. = 1/\phi$ and the GSTABL computer program (Gregory Geotechnical Software, 2003).

The Modified Bishop Method is appropriate where the failure surface is expected to be circular. The Janbu Method is appropriate where the failure surface is expected to be planar.

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mode of failure is circular or planar, the Modified Bishop Method (or equivalent) or the Janbu Method (or equivalent), respectively, shall be used to evaluate the overall stability of the slope. Line, point, or area loads due to shallow foundations located within the soil mass defined by the failure surface shall be included in the analysis.

10.6.2.6 Bearing Resistance At The Service Limit State

The following shall supplement A10.6.2.6.

The settlement of spread footings shall be estimated in accordance with A10.6.2.4 and D10.6.2.4 and compared to the movement criteria established in D10.5.2.2.

10.6.3 Strength Limit State Design**10.6.3.1 Bearing Resistance Of Soil***10.6.3.1.1 General*

The following shall supplement A10.6.3.1.1.

The Department's preferred method for predicting bearing resistance of soils under footings is the theoretical method given in D10.6.3.1.2.

*10.6.3.1.2 Theoretical Estimation**10.6.3.1.2a Basic Formulation*

The following replaces the 5th paragraph and equation A10.6.3.1.2a-1 of A10.6.3.1.2a.

The bearing resistance at the strength limit state (for general shear failure) may be estimated using the following relationship for continuous footings (i.e., $L > 5B$):

$$q_n = cN_c + 0.5\gamma B N_\gamma + \gamma D_f N_q \quad (10.6.3.1.2a-1)$$

A modified form of the general bearing capacity equation may be used to account for the effects of footing shape, ground surface slope and inclined loading as follows:

$$q_{ult} = cN_{cs}i_c + 0.5\gamma B N_{\gamma s}i_\gamma + \gamma D_f N_{qs}i_q \quad (10.6.3.1.2a-10P)$$

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If the foundation within the soil mass defined by the failure surface is supported by deep foundations, i.e. piles or drilled shafts, that bear in material below the failure surface, it is not necessary to include loads due to those foundations in the analysis.

C10.6.3.1.2a

The following shall supplement AC10.6.3.1.2a.

The three modes of shear failure (general, local, and punching) are shown in Figure C10.6.3.1.2a-2P.

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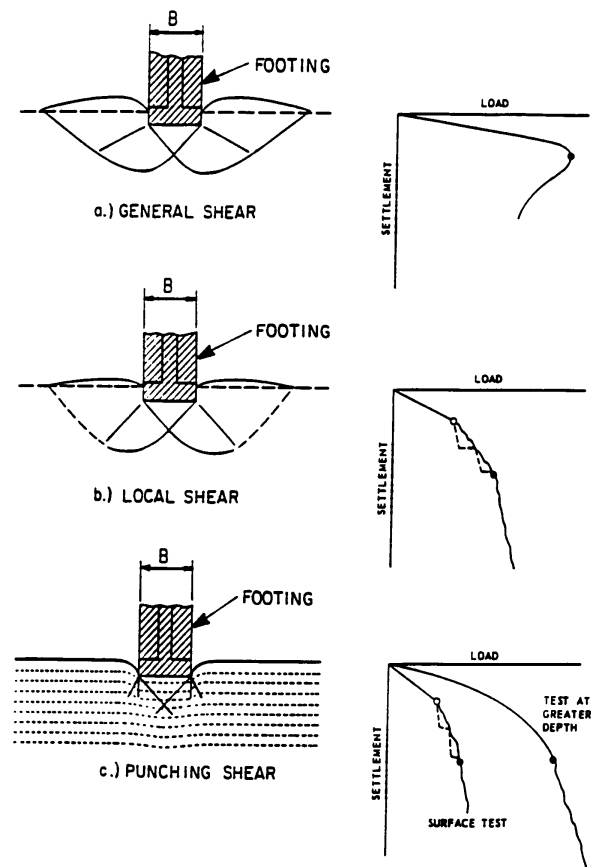


Figure C10.6.3.1.2a-2P Bearing Capacity Failure Modes for Footings on Soil (Vesic 1963).

General shear failure is characterized by a well-defined failure surface extending to the ground surface and is accompanied by sudden rotation and tilting of the footing and bulging of soil on both sides of the footing. General shear failure occurs in relatively incompressible soil and in saturated normally consolidated clays in undrained loading.

Local shear and punching shear are described in AC10.6.3.1.2b.

The failure mode for a particular footing depends primarily on the compressibility of the soil and the footing depth. The relationship between footing depth, mode of failure, and relative density for footings in sand is shown in Figure C10.6.3.1.2a-3P.

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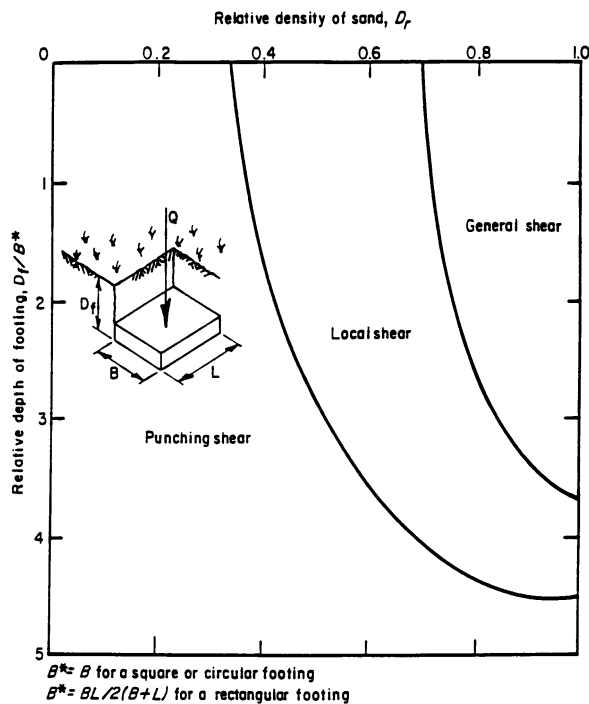


Figure C10.6.3.1.2a-3P Modes of Bearing Capacity Failure for Footings in Sand.

The equation for the ultimate bearing capacity for the case of general shear (Eq D10.6.3.1.2a-1) includes three separate terms. The first is a function of the soil shear strength (ϕ_f and c), the second is a function of the footing width (B) and the third is a function of the footing depth (D_f) and soil unit weight (γ).

The general equation for theoretical estimation of ultimate bearing capacity (Eq D10.6.3.1.2a-1) is an approximate, semi-empirical equation which assumes an infinitely long strip footing with a horizontal base carrying a concentric, vertical load, bearing well above the water table in a homogeneous, plastic soil with a horizontal ground surface.

Numerous investigators have developed bearing capacity factors (N_c , N_γ and N_q) for use in the general bearing capacity equation. The indicated values are those developed by Prandtl and Reissner for N_c and N_q , and those developed by Caquot and Kerisel for N_γ , which are generally considered the most reliable and are currently the most widely used by practicing engineers, Hunt (1986). The values shown in Table A10.6.3.1.2a-1 can also be calculated using the following relationships:

$$N_q = \left(e^{\pi \tan \phi_f} \right) \tan^2 \left(45^\circ + \phi_f / 2 \right) \quad (\text{C10.6.3.1.2a-1P})$$

$$N_c = (N_q - 1) \cot \phi_f \text{ (for } \phi_f > 0^\circ \text{)} \quad (\text{C10.6.3.1.2a-2P})$$

$$N_c = 2 + \pi \text{ (for } \phi_f = 0^\circ \text{)} \quad (\text{C10.6.3.1.2a-3P})$$

$$N_\gamma = 2(N_q + 1) \tan \phi_f \quad (\text{C10.6.3.1.2a-4P})$$

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s_c, s_γ, s_q = For footing shapes other than continuous footings (i.e., $L < 5B$), footing shape correction factors as specified in Table A10.6.3.1.2a-3 (dim). For $L \geq 5B$, shape factors equal 1.0.

10.6.3.1.2c Considerations for Footings on Slopes

The following shall supplement A10.6.3.1.2c.

For footings located on slopes or within $3B$ of a slope crest, q_n may be determined using the following revised version of Equation D10.6.3.1.2a-10P.

$$q_n = cN_{cq}s_{ci} + 0.5\gamma BN_{\gamma q}s_{\gamma i} \quad (10.6.3.1.2c-4P)$$

Refer to Figure A10.6.3.1.2c-1 for values of N_{cq} for footings in cohesive soils and Figure A10.6.3.1.2c-2 for values of $N_{\gamma q}$ for footings in cohesionless soils.

COMMENTARY

The shape factors are semi-empirical factors based on load tests of footings with various shapes.

Depth factors are not included in Equations D10.6.3.1.2a-1 or D10.6.3.1.2a-10P, which treats all soil above the footing bearing level as a surcharge load, and neglects the shearing resistance of the overburden along the failure surface. Depth factors to account for overburden shearing resistance have been developed by several investigators and could be applied where the soil above the footing is expected to provide significant shear resistance. However, Vesic notes that there is evidence that the additional shearing resistance is negligible for backfilled footings or footings in compressible overburden and should, therefore, be neglected, Winterkorn and Fang (1975).

The effect of an inclined load is to induce a horizontal component in the foundation reaction, such that failure of the footing may occur by either general shear (bearing resistance) or sliding. Inclined loads have the effect of reducing the bearing capacity of a footing.

C10.6.3.1.2c

The following shall supplement AC10.6.3.1.2c.

The effect of a sloping ground surface in front of a footing or an inclined footing base is to reduce bearing resistance. The reduction in bearing capacity is due to the decreased volume of soil providing passive resistance to failure on the downslope side of the footing. The modified bearing capacity factors (N_{cq} and $N_{\gamma q}$) are based on the stability number (N_s) for cohesive soil and the effective stress angle of friction (ϕ_t) for cohesionless soil or drained analysis of cohesive soil. For cohesive soils with $\phi = 0$, $N_{\gamma q} = 1.0$. For cohesionless soils with $c = 0$, N_{cq} is not needed. The ground slope factors do not account for pre-existing shearing stresses in the ground which, in combination with footing loads, may cause a slope failure.

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10.6.3.1.2d Considerations for Two-Layer Soil Systems - Critical Depth

The following shall supplement A10.6.3.1.2d.

If the soil profile is layered, the general bearing capacity equation shall be modified to account for differences in failure mode between the layered case and the homogeneous soil case assumed in Equation D10.6.3.1.2a-10P.

10.6.3.1.2e Two-layered Soil System in Undrained Loading

The following shall replace the first paragraph of A10.6.3.1.2e.

Where a footing is supported on a two-layered soil system subjected to undrained loading, the nominal bearing resistance may be determined using Equation D10.6.3.1.2a-1 with the following modifications:

$$q_n = c_1 N_m + \gamma D_f \quad (10.6.3.1.2e-4P)$$

10.6.3.1.2gP Groundwater

Nominal bearing resistance shall be determined using the highest anticipated groundwater level at the footing location. The effect of ground water level on the ultimate bearing resistance shall be considered by using a weighted average soil unit weight in Equation D10.6.3.1.2a-10P. If $\phi_f < 37^\circ$, the following equations may be used to determine the weighted average unit weight:

$$\text{for } z_w \geq B: \text{ use } \gamma = \gamma_m \text{ (no effect)} \quad (10.6.3.1.2gP-1)$$

$$\text{for } z_w < B: \text{ use } \gamma = \gamma' + (z_w/B)(\gamma_m - \gamma') \quad (10.6.3.1.2gP-2)$$

$$\text{for } z_w \leq 0: \text{ use } \gamma = \gamma' \quad (10.6.3.1.2gP-3)$$

If $\phi_f \geq 37^\circ$, the following equations may be used to determine the weighted average unit weight:

$$\text{for } z_w \geq D: \text{ use } \gamma = \gamma_m \text{ (no effect)} \quad (10.6.3.1.2gP-4)$$

$$\text{for } z_w < D: \gamma = (2D - z_w)(z_w \gamma_m / D^2) + (\gamma' / D^2)(D - z_w)^2 \quad (10.6.3.1.2gP-5)$$

$$D = 0.5B \tan(45^\circ + \phi_f/2) \quad (10.6.3.1.2gP-6)$$

$$\text{for } z_w \leq 0: \text{ use } \gamma = \gamma' \quad (10.6.3.1.2gP-7)$$

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C10.6.3.1.2dP

Bearing failure of spread footing foundations on layered cohesive soils occurs by two principal modes depending on whether the bearing stratum is underlain by a stiffer or softer soil unit. For cases where the soft layer overlies a stiffer layer, failure partially occurs by lateral flow of the soft soil layer. For cases where the stiff layer overlies a soft layer, failure usually occurs by punching of the stiffer soil into the underlying softer soil.

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Refer to Figure 10.6.3.1.2gP-1 for definition of terms used in these equations.

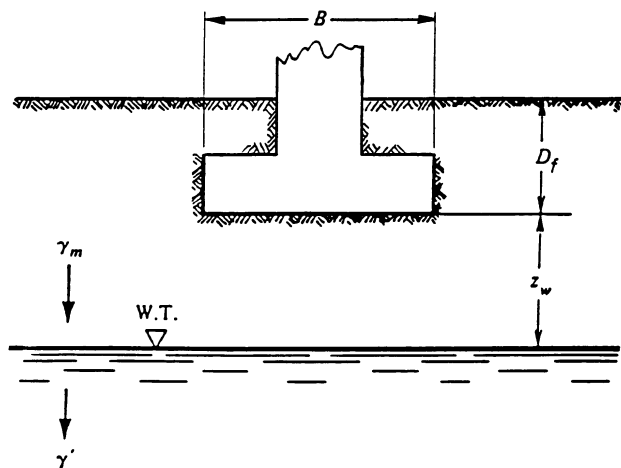


Figure 10.6.3.1.2gP-1 - Definition Sketch for Influence of Groundwater Table on Bearing Capacity

10.6.3.1.2hP Footing Base Inclination

Inclined footing bases shall not be used without the approval of the Chief Bridge Engineer.

10.6.3.2 Bearing Resistance of Rock

10.6.3.2.1 General

The following shall supplement A10.6.3.2.1.

Rock masses may also pose special problems that shall be considered in design. Typical examples of special problems include weathering, chemical effects, solutioning and subsidence. See Publication 293M, "Geotechnical Engineering Manual", to identify the presence and extent of these special problem conditions throughout the Commonwealth.

10.6.3.2.2 Semiempirical Procedures

The following shall replace A10.6.3.2.2.

The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

The design of footings on rock shall account for the condition and spacing of joints and other discontinuities. The ultimate bearing resistance of footings on rock can be estimated using the following relationship:

$$q_n = N_{ms} C_o \quad (10.6.3.2.2-1P)$$

Refer to Table 10.6.3.2.2-1P for values of N_{ms} . Values of C_o should be determined from the results of laboratory

COMMENTARY

C10.6.3.2.2

The following shall replace AC10.6.3.2.2.

The bearing resistance of a footing on jointed or broken rock is dependent on the relationship between the joint spacing, the footing geometry and the condition of the joints.

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988).

$$q_n = \left[\sqrt{s} + \left(m \sqrt{s} + s \right)^{\frac{1}{2}} \right] C_o \quad (C10.6.3.2.2-1P)$$

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testing of rock core obtained within 2B of the base for square and circular footings and within 4B of the base of the footing for footing where $L/B \geq 5$. Where rock strata within this interval are variable in strength, the rock with the lowest capacity should be used to determine q_n . As a guide, Table 10.6.3.2.2-2P can be used to estimate C_o . For rocks defined by very poor quality, the value of q_n should be determined as the value of q_n for an equivalent soil mass.

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This procedure is based on the unconfined compressive strength of the intact rock core sample.

The terms s and m are constants from Table A10.4.6.4-4. Values of the term in brackets (designated as N_{ms}) as a function of rock type and quality are presented in Table 10.6.3.2.2-1P, such that q_n can be determined using Equation C10.6.3.2.2-1P.

Table 10.6.3.2.2-1P - Values of Coefficient N_{ms} for Estimation of the Nominal Bearing Resistance of Footings on Broken or Jointed Rock, Modified after Hoek (1983)

ROCK MASS QUALITY	GENERAL DESCRIPTION	RMR ⁽¹⁾ RATING	RQD ⁽²⁾ (%)	N_{ms} ⁽³⁾				
				A	B	C	D	E
Excellent	Intact rock with joints spaced >10 ft. apart	100	95-100	3.8	4.3	5.0	5.2	6.1
Very Good	Tightly interlocking, undisturbed rock with rough unweathered joints spaced 3 to 10 ft. apart	85	90-95	1.4	1.6	1.9	2.0	2.3
Good	Fresh to slightly weathered rock, slightly disturbed with joints spaced 3 to 10 ft. apart	65	75-90	0.28	0.32	0.38	0.40	0.46
Fair	Rock with several sets of moderately weathered joints spaced 1 to 3 ft. apart	44	50-75	0.049	0.056	0.066	0.069	0.081
Poor	Rock with numerous weathered joints spaced 1 to 20 in. apart with some gouge	23	25-50	0.015	0.016	0.019	0.020	0.024
Very Poor	Rock with numerous highly weathered joints spaced < 2 in. apart	3	<25	Use q_{ult} for an equivalent soil mass				

⁽¹⁾ Geomechanics Rock Mass Rating (RMR) System, in accordance with A10.4.6.4

⁽²⁾ Range of RQD values provided for general guidance only; actual determination of rock mass quality should be based on RMR.

⁽³⁾ Value of N_{ms} as a function of rock type; refer to Table 10.6.3.2.2-2P for typical range of values of C_o for different rock types in each category

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Table 10.6.3.2.2-2P - Typical Range of Uniaxial Compressive Strength (Co) as a Function of Rock Category and Rock Type

ROCK CATEGORY	GENERAL DESCRIPTION	ROCK TYPE	Co ⁽¹⁾
			ksf
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	600-6,400
		Limestone	600-6,000
		Carbonatite	800-1,400
		Marble	800-5,000
		Tactite-Skarn	2,800-6,800
B	Lithified argillaceous rock	Argillite	600-3,000
		Claystone	30-170
		Marlstone	1,000-4,000
		Phyllite	600-5,000
		Siltstone	200-2,400
		Shale ⁽²⁾	150-740
		Slate	3,000-4,400
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	600-4,600
		Sandstone	1,400-3,600
		Quartzite	1,200-8,000
D	Fine-grained igneous crystalline rock	Andesite	2,000-3,800
		Diabase	450-12,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,400-5,800
		Gabbro	2,600-6,400
		Gneiss	500-6,400
		Granite	300-6,800
		Quartzdiorite	200-2,000
		Quartzmonzonite	2,800-3,400
		Schist	200-3,000
		Syenite	3,800-9,000

⁽¹⁾ Range of Uniaxial Compressive Strength values reported by various investigations

⁽²⁾ Not including oil shale

10.6.3.2.3 Analytic Method

The following shall replace A10.6.3.2.3.

The nominal bearing resistance of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters determined in accordance with A10.4.6.4. The influence of discontinuities on the failure mode shall also be considered.

The bearing capacity of a footing on jointed or broken rock is dependent on the relationship between the joint spacing and the footing geometry, and the condition of the joints as described below.

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10.6.3.2.3aP Footings on Closely-Spaced Joint Sets

Closely-spaced joints are discontinuities with a spacing less than the minimum plan dimension of the footing. Footings on closely spaced joint sets may fail either by uniaxial compression or shear, depending on the condition and orientation of the joints.

- Open Joints

Open joint sets are defined as discontinuities across which load cannot be transferred due to the presence of an open space or soft joint filler. For open joints, failure is likely to occur by uniaxial compression of the rock columns. The nominal bearing resistance of the rock mass may be determined for this case by the following relationship:

$$q_n = 2K_e c \tan(45 + \phi_{fm}/2) \quad (10.6.3.2.3aP-1)$$

In determining q_n , strength parameters representing rock mass shall be used. For cases in which the parameters are determined from intact samples rather than the rock mass, the value of c , obtained from intact rock, shall be reduced by the factor K_e , using the following relationship from Gardner (1987):

$$K_e = 0.0231(RQD) - 1.32 \geq 0.15 \quad (10.6.3.2.3aP-2)$$

Values of the rock mass angle of friction (ϕ_{fm}) are typically 50 to 75% of those of the intact material, Kulhawy and Goodman (1980). Values of c and ϕ_{fm} for the intact rock can be determined from the results of triaxial compression tests on rock core samples. For poor quality rock in which intact samples cannot be obtained, use of the pressuremeter test may be considered for determining the in situ rock properties. When in situ testing or triaxial compression testing is not or cannot be performed, the nominal bearing capacity of the rock mass can be estimated by assuming c equal to approximately 5 to 10% of C_o , obtained from the results of uniaxial compressive strength or point load tests, and assuming ϕ_{fm} equal to zero as presented in Kulhawy and Goodman (1980).

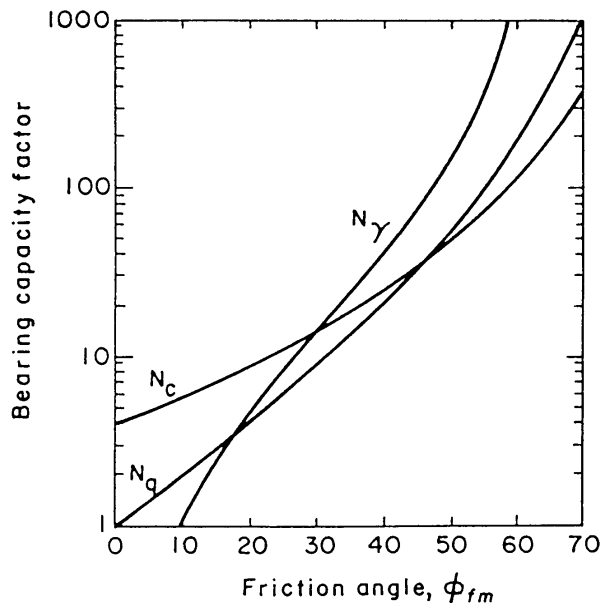
- Closed Joints

Closed joint sets are defined as discontinuities across which load can be transferred because of contact between rock surfaces on either side of the joints. For closed joints, failure is likely to occur by general shear failure of the rock across the joint sets. For this case, the nominal bearing capacity shall be determined using Equation D10.6.3.1.2a-1 and the bearing capacity factors as defined in Figure 10.6.3.2.3aP-1. Where applicable, terms in this equation shall be modified by factors described in A10.6.3.1.2e, A10.6.3.1.2f, D10.6.3.1.2e and D10.6.3.1.2gP. Bearing capacity factors shall be developed using a value of ϕ_{fm}

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adjusted to account for rock mass characteristics.



Note:

ϕ_{fm} = angle of internal friction
of rock mass

Figure 10.6.3.2.3aP-1 - Bearing Capacity Factors for Development of General Wedge Shear Zone for Footings on Rock with Closed Joints

10.6.3.2.3bP Footings on Widely-Spaced Joint Sets

Widely-spaced joints are discontinuities with a spacing greater than the minimum plan dimension of the footing. Where wide joint sets are present, failure occurs by splitting of rock below the footing, which ultimately leads to general shear. For square and circular footings, the nominal bearing capacity can be determined by the following relationship, Kulhawy and Goodman (1980):

For circular footings:

$$q_{ult} = J(K_e c) N_{cr} \quad (10.6.3.2.3bP-1)$$

For square footings:

$$q_{ult} = 0.85J(K_e c) N_{cr} \quad (10.6.3.2.3bP-2)$$

See Figure 10.6.3.2.3bP-1 for values of J and N_{cr} to use in the equations. The bearing capacity factor shall be developed using a value of ϕ_{fm} adjusted to account for rock mass characteristics as discussed in D10.6.3.2.3aP. If the shear strength of the rock is not or cannot be determined by in-situ testing or triaxial testing of rock core samples, the bearing capacity can be conservatively estimated by

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assuming ϕ_{fm} equal to zero and c equal to approximately 5 to 10% of C_o .

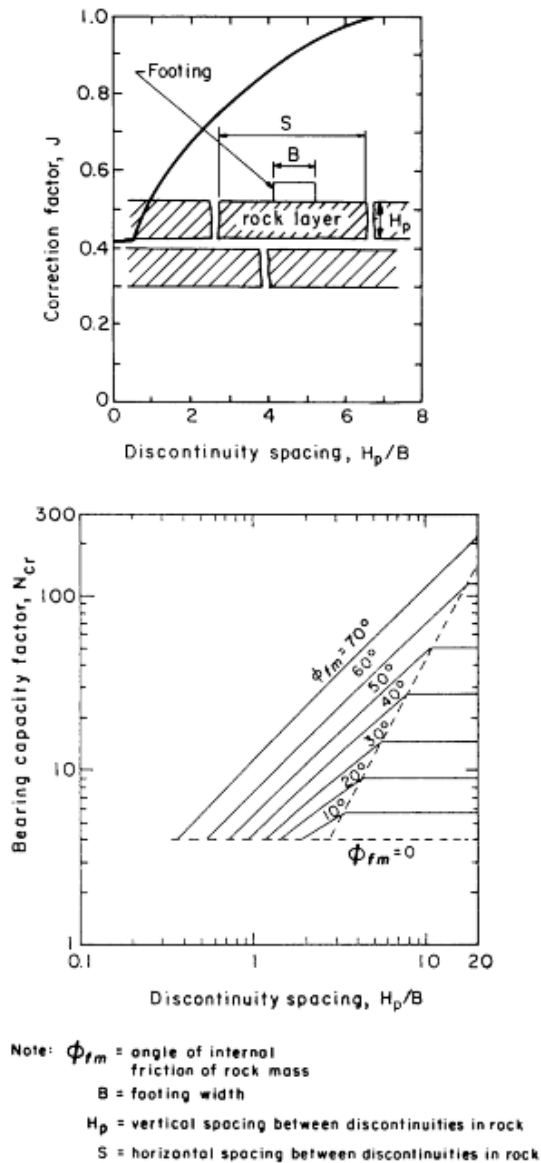


Figure 10.6.3.2.3bP-1 - Bearing Capacity Factors for Footings on Rock with Widely-Spaced Joint Sets

10.6.3.2.3cP Footings on Rigid Layer Over Weaker Layer

For a thick rigid layer overlying a weaker layer, failure will probably occur by flexure of the upper layer where the flexure strength of unfractured rock is approximately 10 to 20% of q_n . For a thin rigid layer overlying a weaker layer, failure will probably occur by tensile failure of the upper layer where the tensile strength of unfractured rock is approximately 5 to 10% of q_n .

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10.6.3.4 Failure by Sliding

The following shall replace equation 10.6.3.4-1 in A10.6.3.4.

$$R_R = \phi R_n = \phi_\tau R_\tau \quad (10.6.3.4-1)$$

The following shall supplement A10.6.3.4.

The nominal resistance for sliding for foundations on soils exhibiting both frictional and cohesive shear strength components, i.e., "c- ϕ " soils, may be taken as:

$$R_\tau = V \tan \delta + c_a B' \quad (10.6.3.4-3P)$$

where:

V = total vertical force per unit width (kips/ft)

$\tan \delta$ = $\tan \phi_f$ for concrete cast against soil
 = $0.8 \tan \phi_f$ for a precast concrete footing

c_a = adhesion between footing and soil, taken as $c(0.21 + 0.54/c \leq 1.0)$, unless better data is available (ksf)

B' = effective footing width as specified in A10.6.1.3 (ft.)

Passive resistance shall be neglected in evaluation of sliding failure, unless the base of the footing extends below the depth of maximum scour, freeze-thaw or other disturbances. In the latter case only, the embedment below the greater of these depths may be considered effective.

The passive resistance should be neglected if the soil providing passive resistance is soft, loose or disturbed, or if the contact between the soil and footing is not tight.

COMMENTARY

C10.6.3.4

The following shall supplement AC10.6.3.4.

Engineering judgment should be exercised when making the determination of whether or not to count on the cohesive shear strength of a c- ϕ soil. A c- ϕ analysis should only be performed on soils which exhibit a true effective stress cohesion strength under long-term, drained conditions. The drained, effective stress strength parameters for this condition should be determined using a minimum of three shear strength tests or three undrained tests with pore pressure measurements.

In cases where the passive pressure is used, the resultant force shall be applied at $0.4 H$, where H is the height of the effective soil in front of the footing.

Unacceptable deformations may occur before passive resistance is mobilized.

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10.6.5 Structural Design

The following shall supplement A10.6.5.

The actual distribution of contact pressure for a rigid footing with eccentric loading about one axis is shown in Figure 10.6.5-1P.

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C10.6.5

The following shall supplement AC10.6.5

In reality, the contact pressure is nonlinear due to the flexibility of the footing which tends to reduce the value of $q_{\max}$ to a value less than the high toe contact pressure associated with a rigid footing.

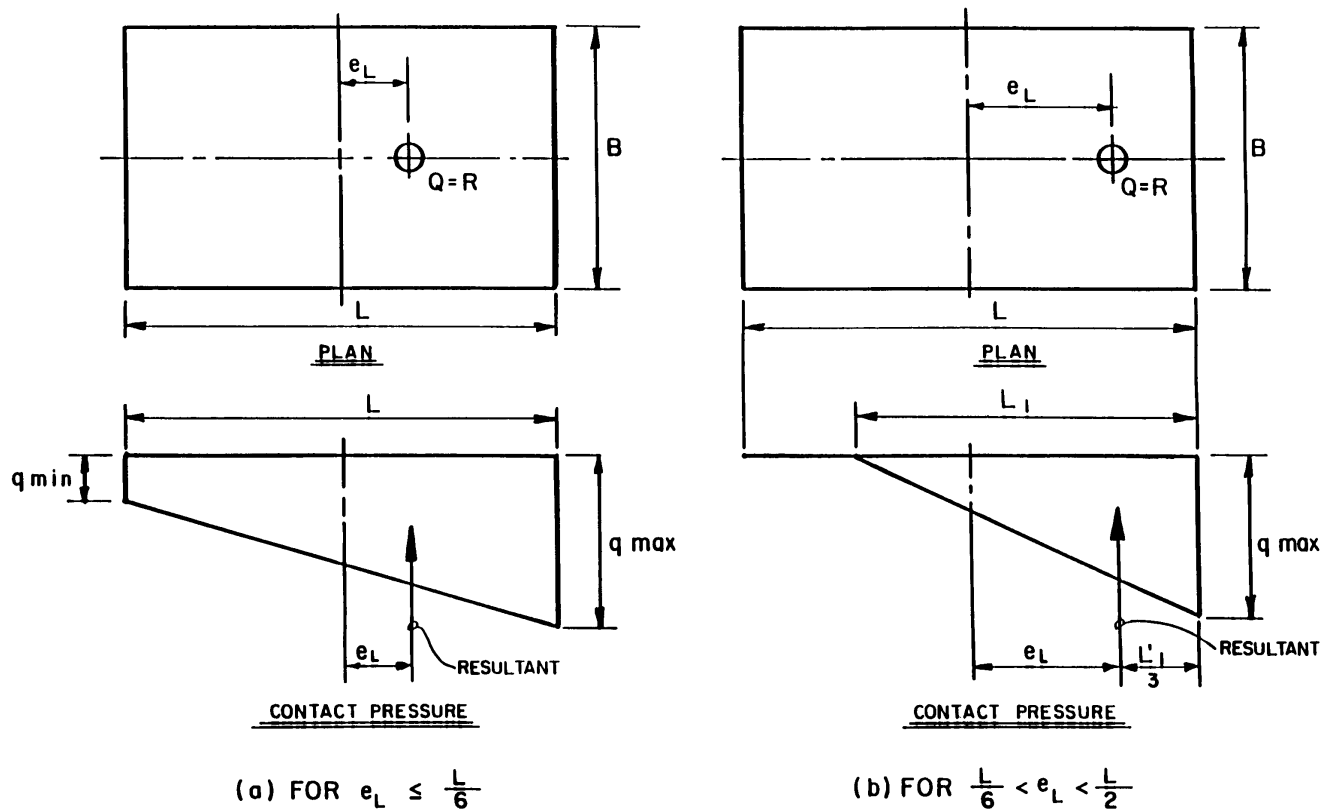


Figure 10.6.5-1P – Contact Pressure for Footing Loaded Eccentrically about One Axis

For an eccentricity (e_L) in the L direction, the actual maximum and minimum contact pressures may be determined as follows:

for $e_L < L/6$:

$$q_{\max} = V \left[1 + \left(6e_L / L \right) \right] / BL \quad (10.6.5-1P)$$

$$q_{\min} = V \left[1 - \left(6e_L / L \right) \right] / BL \quad (10.6.5-2P)$$

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for $L/6 < e_L < L/2$:

$$q_{\max} = 2V / \left(3B \left[(L/2) - e_L \right] \right) \quad (10.6.5-3P)$$

$$q_{\min} = 0 \quad (10.6.5-4P)$$

$$L_1 = 3 \left[(L/2) - e_L \right] \quad (10.6.5-5P)$$

For an eccentricity (e_B) in the B direction, the maximum and minimum contact pressures may be determined using Equations 10.6.5-1P through 10.6.5-5P by replacing terms labeled L by B, and terms labeled B by L.

The actual distribution of contact pressure for rigid footings with eccentric loading about both axes are shown in Figure 10.6.5-2P.

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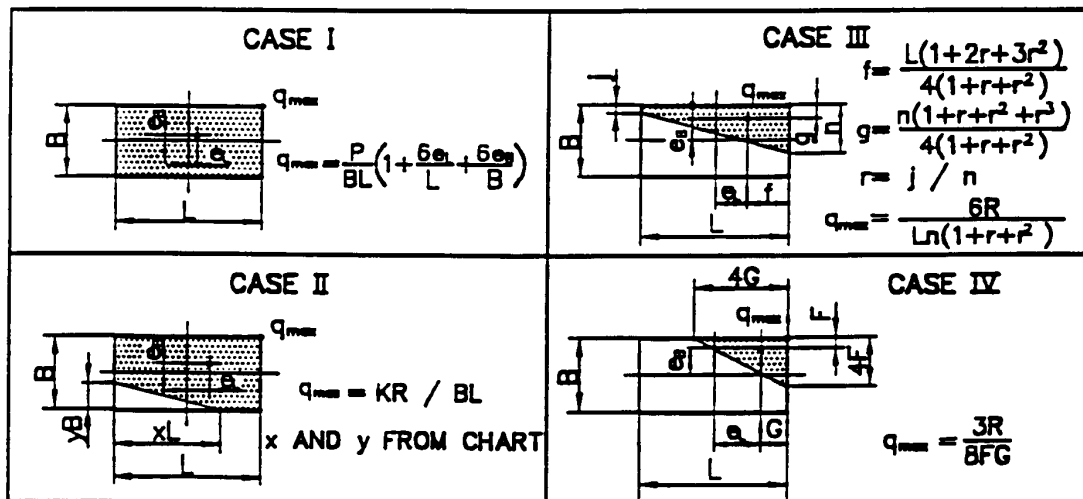
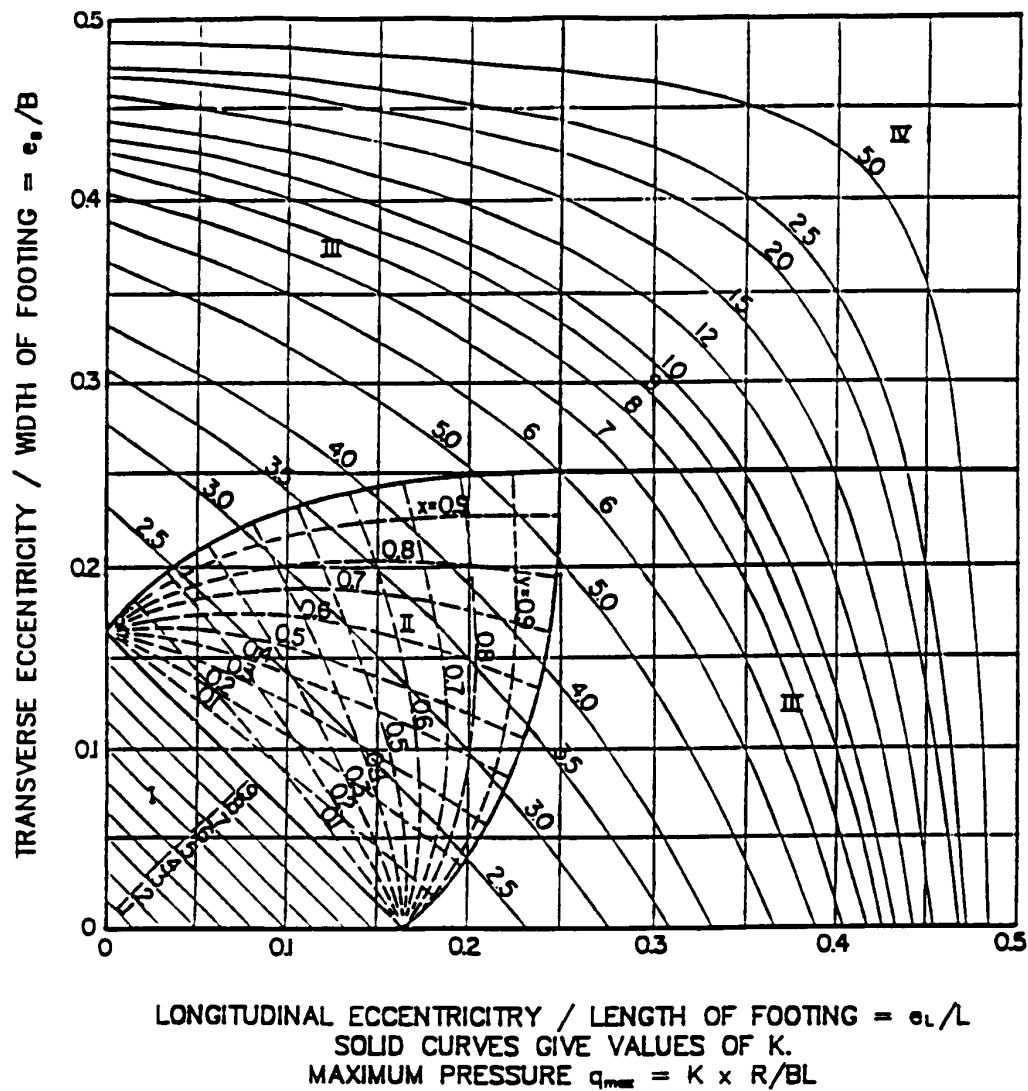


Figure 10.6.5-2P - Contact Pressure for Footing Loaded Eccentrically about Two Axes, Modified after AREA (1980)

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10.6.5.1P Unreinforced Concrete Footing

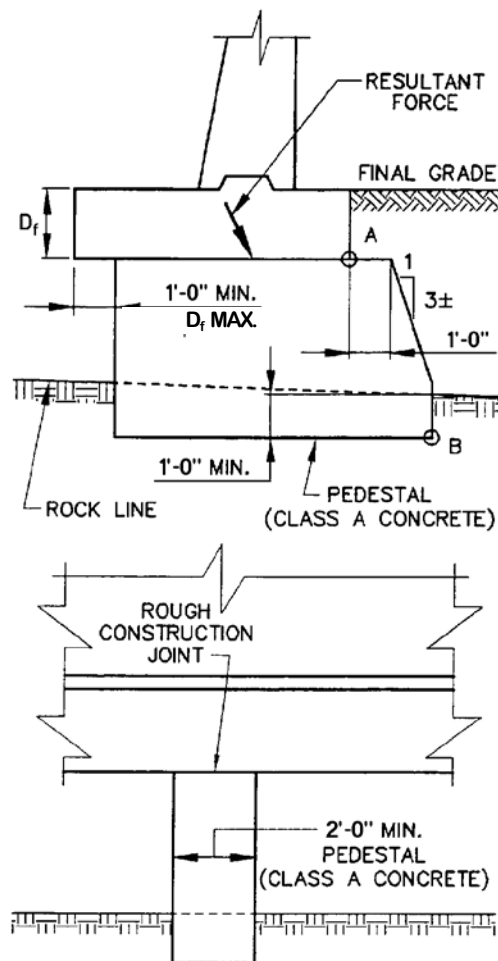
Special approval from the Chief Bridge Engineer is needed to permit plain cement concrete footings.

10.6.5.2P Foundation Pedestals

The thickness of foundation pedestals shall be a minimum of 24 in. Sufficient longitudinal reinforcement shall be provided at the top and bottom faces of the footing slab for continuous beam action between pedestals. Additional dowels may be required for integral action between the stem and footing. If required to ensure integral action between footings and pedestals, transverse shear keys and dowels (No. 6 x 4 ft. at approximately 18 in. c/c each face) shall be provided. See Figure 10.6.5.2P-1 for other design details.

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- (1) FOOTING AND PEDESTAL DIMENSIONS SHALL SATISFY THE FOLLOWING REQUIREMENTS:
- (a) EVALUATION OF OVERTURNING (ABOUT A FOR FOOTING AND ABOUT B FOR PEDESTALS) SHALL BE IN ACCORDANCE WITH A11.6.3.3.
 - (b) THE RATIO OF SERVICE I VERTICAL LOADS TO THE SERVICE I HORIZONTAL LOADS SHALL NOT BE LESS THAN 2.0.
 - (c) THE EFFECT OF LATERAL EARTH PRESSURE ON THE PEDESTAL CAN BE IGNORED.
- (2) THE MINIMUM CLEAR DISTANCE BETWEEN PEDESTALS SHALL BE APPROXIMATELY 10 ft.

Figure 10.6.5.2P-1 - Foundation Pedestal

10.6.6P Foundation Submission

As part of the foundation approval process, a foundation submission letter shall be submitted to the Department for use in foundation review in compliance with PP1.9.4 and D11.4.4P. The letter shall include the basis for foundation capacity determination and the relevance of field conditions and construction procedures to develop the foundation capacity.

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10.7 DRIVEN PILES

The following shall supplement A10.7.

This article presents design specifications for driven piles. Refer to Article A10.9 and D10.9 for design specifications for micropiles.

10.7.1 General**10.7.1.1 Application**

The following shall supplement A10.7.1.1.

Piles must be sufficiently embedded below theoretical scour depth (refer to PP7.2 to determine the estimated scour depth) to properly support the structure. Where unsuitable material is less than 10 ft. in depth, consideration shall be given to the economics of spread footings supported on compacted structure backfill material as an alternate to piles.

Preloading of compressible soil by embankment surcharge may be used to reduce settlement to permit use of spread footings, or to reduce negative friction loading on piles. The cost of preloading compared to pile foundations shall be evaluated and submitted with the Foundation Design Report.

10.7.1.2 Minimum Pile Spacing, Clearance, and Embedment into Cap

The following shall replace A10.7.1.2.

Footings shall be proportioned so the center-to-center pile spacing is not less than 3 ft. or

- 2.5 times the pile diameter for round or octagonal piles
- 2.5 times the diagonal dimension for H-piles used in integral abutments
- 2.0 times the diagonal dimension for square piles or H-piles (not used in integral abutments)

Maximum pile spacing shall be 15 ft. The distance from the edge of the footing to the nearest side of any pile shall be a minimum of 9 in., and not less than 1'-6" from the centerline of pile.

Piles shall be embedded at least 1 ft. into footings, or 1'-6" into stub abutments supported by a single row of piles. For details and reinforcement bar locations, see Figure 10.7.1.2-1P. Minimum footing thickness shall be 2'-6".

Reinforcement bar and pile locations shall be as follows:

- Middle and/or rear piles aligned with the front row piles for ease of construction

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- Reinforcement bars placed evenly between piles and hooked for the reinforcement provided transversely in between piles
- Cutting of flanges or rebars to accommodate reinforcement shall not be allowed (except that holes may be burned through piles within the embedment zone to accommodate steel reinforcement required for seismic loading)
- For main reinforcement bars near piles in the footing, a clearance of 1 in. is recommended. Reinforcement bar spacing near piles should be adjusted accordingly.

If piles are out of position more than 6 in. preventing proper placement of reinforcement bars, the Contractor will have the option of either placing the reinforcement bars on top of the piles and providing additional thickness at no additional cost to the Department, or providing bottom reinforcement bars in such a way (splayed or spliced) to provide adequate structural strength, if approved by the District Bridge Engineer.

Contractors may be permitted to redesign pile footings to provide the bottom reinforcement bars above piles at no additional cost to the Department, provided the following requirements are met:

- Meet the same strength and serviceability requirements appropriate for the original design
- Minimum 3 ft. thick footing
- Minimum earth cover above the top of the footing, as shown on the original plan
- Maintain the original design bottom of footing elevation in a scour environment
- Meet seismic requirements in accordance with AASHTO Appendix A11.

For Integral abutments, refer to Appendix G and BD-667M.

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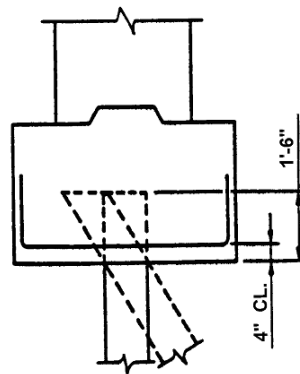
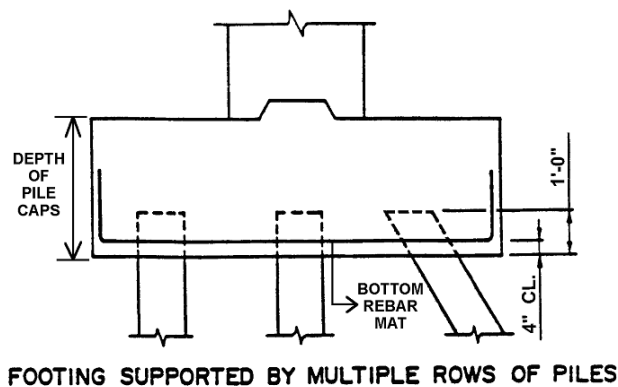


Figure 10.7.1.2-1P - Minimum Projection of Piles into Cap and Clearance to Bottom Reinforcement Bars

10.7.1.3 Piles Through Embankment Fill

The following shall supplement A10.7.1.3.

The minimum penetration length of piles shall be 10 ft. unless otherwise approved by the Chief Bridge Engineer for a specific project.

For piles through embankment fill, the plans shall specify that select fill be placed and compacted to the footing elevation before pile driving. The stiffest cost-effective pile with tip reinforcement shall be used to facilitate embankment penetration without auguring or predrilling. Auguring to the original groundline will be permitted, if necessary. Where point-bearing or end bearing piles extend through fill and an underlying compressible soil deposit, the effects of negative friction (downdrag) on the pile shall be considered (see A10.7.3.7)

10.7.1.4 Batter Piles

The following shall supplement A10.7.1.4.

Where ground settlements of greater than 1/4 in. are estimated, batter piles should be avoided or installed after the settlement is substantially complete. Otherwise, the effects of pile bending due to downdrag forces shall be

C10.7.1.3

The following shall supplement AC10.7.1.3.

See D10.7.1.6.5P for further discussion of predrilling.

For integral abutments 10 ft. minimum pile penetration must neglect predrilled portion.

C10.7.1.4

The following shall supplement AC10.7.1.4.

Batter piles attract lateral load in groups which include vertical piles due to their greater resistance to lateral deformation. Therefore, where a combination of batter and vertical piles is subjected to seismic loading, special

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evaluated during design.

Batter piles may be used in pile groups to reduce bending in vertical piles and thereby maximize their axial load capacity. Batter piles are recommended for abutments and retaining walls. The bearing resistance of a pile group containing batter piles may be determined by treating the batter piles as vertical piles.

Layout of batter piles shall be arranged so that batter piles will not interfere with each other below the pile cap, or with piles below adjacent pile caps.

In general, a pile batter of 1 (horizontal) on 4 (vertical) is considered desirable, but in cases where sufficient lateral resistance is not otherwise attainable, a batter of 1 on 3 may be specified. In no case shall the batter exceed 1 on 3. The Engineer should consider the slope of top of bedrock when setting pile batter.

10.7.1.5 Pile Design Requirements

The following shall supplement A10.7.1.5.

The load supporting resistance shall be determined using current Department practice according to applicable general notes in PP1.7.5 and Publication 408, Section 1005. The pile type, load resistance and driving method shall be as approved by the Chief Bridge Engineer. Refer to A10.7.3 and D10.7.3 for methods to determine pile resistance

10.7.1.6 Determination of Pile Loads*10.7.1.6.2 Downdrag*

The following shall supplement A10.7.1.6.2.

The effects of negative skin friction can be ignored for point bearing piles if effective measures are taken to reduce effects of negative skin friction. If downdrag is considered, use the minimum value for γ_p as specified in Table A3.4.1-2.

Negative skin friction loads that cause excessive settlement may be reduced by measures such as:

- application of bitumen or other viscous coating to the pile surfaces before installation,
- inducing or permitting consolidation settlement to occur prior to pile installation, or
- isolating pile from surrounding settling soil by installing the pile through casing.

10.7.1.6.3 Uplift Due to Expansive Soils

The following shall supplement A10.7.1.6.3.

Piles driven in swelling soils may be subjected to uplift forces in the zone of seasonal moisture change. Uplift loads can be reduced by application of bitumen or other viscous

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consideration should be given to the distribution of load among piles and the design of the pile-cap connection.

Settlement induces bending moments in the shafts of batter piles (Tomlinson, 1987). No documented procedure is available to analyze the behavior of batter piles subjected to downdrag forces. A rational method of analysis for bending of batter piles should evaluate the piles as beams on an elastic foundation and that the piles assume deflected shape of the soil mass. The analysis should consider the effects of pile type and geometry, the length of pile within the compressible soil zone, the initial vertical and lateral loading at the pile cap, the pile fixity, and the lateral subgrade reaction between the soil and pile.

It is the Department preference that no more than 50 percent of the back row of piles be battered if it is economical.

C10.7.1.6.2

The following shall supplement AC10.7.1.6.2.

Although the use of bituminous material coating is permitted, environmental concerns and practical application (bitumen loss during driving) often prevent this option from being feasible.

C10.7.1.6.3

The following shall supplement AC10.7.1.6.3.

Based on Department experience, uplift on piles due to swelling soils is generally not a problem in Pennsylvania.

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coatings to the pile surface in the swelling zone.

Tension in piles is not permitted at Service Limit states.

At Strength Limit States, the uplift capacity may be taken as 10 percent of the axial structural capacity.

10.7.1.6.4 Nearby Structures

10.7.1.6.5P Pre-Drilling for Driven Piles

Pre-drilling for driven piles shall be categorized as Mandatory Pre-drilling or Pre-drilling for Unforeseen Obstructions.

10.7.1.6.5aP Mandatory Pre-Drilling

Mandatory Pre-drilling shall be specified in situations that require drilling to obtain the necessary 10-foot pile length, or to obtain a required pile tip elevation by penetrating through upper inadequate rock layers, or vibration mitigation or other situations that require mandatory pre-drilling to obtain a required tip elevation. Pre-drilled piles shall be indicated on the pile layout plans. The contract documents shall include standard special provision ID 10051A. This special provision is a bid item that includes all operations necessary to perform the work including mobilization, access to the foundations, drilling, maintaining an open hole, casing and backfilling with aggregate.

This special provision shall not be used as a contingency item in the contract.

The special provision uses a granular material to backfill the hole. For situations that require any special backfill material this material shall be specified in the contract documents.

The special provision indicates the steel casing to be removed, therefore it is temporary casing. If permanent casing is required, the construction plans shall indicate the required wall thickness, the length of the casing and include a bid item for the permanent casing (linear foot).

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C10.7.1.6.4

The following shall supplement AC10.7.1.6.4.

A pre-construction survey of nearby structures to document conditions prior to driving should also be considered.

C10.7.1.6.5P

Pile driving is a very economical construction method. For limited situations, it will be necessary to pre-drill piles to reach the predetermined tip elevations. However, many geotechnical reports and construction contracts specify pre-drilling of piles because of anticipated or perceived difficulties in driven piles obtaining predetermined tip elevations. In the past these construction contracts have included substantial predrilling quantities as a contingency. With the utilization of the special provisions in D10.7.1.6.5aP and D10.7.1.6.5bP in construction contracts, the Department should realize an improvement in bid consistency and resolution of construction issues associated with predrilling.

C10.7.1.6.5aP

Special backfill material may include slurry to be used in limestone formations.

Permanent casing may be used to eliminate downdrag loads on piles.

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10.7.1.6.5bP Pre-Drilling for Unforeseen Obstructions

Pre-drilling for Unforeseen Obstructions shall be specified in situations where it is uncertain if piles can be driven to the predetermined tip elevation due to unforeseen obstructions. Potential obstructions include but are not limited to reinforced concrete, timber, boulders, rock pinnacles or existing piles. The contract documents shall include standard special provisions:

- ID 10052A, Predrilling for Unforeseen Obstructions, Pile Extraction and Re-Driving.
- ID 10053A, Mobilization for Pre-drilling.

Items for the mobilization and pile extraction and redrive are bid as Predetermined Amounts (PDA). Items for Predrilling for Unforeseen Obstructions include earth drilling and obstruction drilling which shall be bid as predetermined minimum contract unit price. The proposal shall indicate \$100/LF for earth drilling and \$500/LF for obstruction drilling. The estimated quantity for earth drilling shall be 10% of the total length of piles at each substructure unit and for obstruction drilling 1% of the total length of piles at each substructure unit.

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C10.7.1.6.5bP

Various scenarios may exist in the field regarding the payment under this special provision. The following four scenarios provide guidance relative to the use of this special provision:

- 1.1 A production pile (with tip reinforcement) is driven but hangs up above tip elevation. [Paid in accordance with Publication 408, Section 1005.]
- 1.2 It is extracted. [Paid for by the item for Pile Extraction and Redrive.]
- 1.3 The hole is drilled to the predetermined pile tip elevation. [Soil drilling is to be paid at \$100/LF and obstruction drilling is to be paid at \$500/LF per the special provision.]
- 1.4 Original pile (accepted by engineer) is placed in the hole, backfilled, and driven to refusal at tip elevation. [Driven pile is paid for by the item for Pile Extraction and Redrive.]
- 2.1 A production pile is driven but hangs up above tip elevation. [Paid in accordance with Publication 408, Section 1005.]
- 2.2 It is extracted and discarded. [Paid for by the item for Pile Extraction and Redrive. The material cost of the discarded pile is in addition to the pile extraction and redrive operations.]
- 2.3 The hole is drilled to the predetermined pile tip elevation. [Soil drilling is to be paid at \$100/LF and obstruction drilling is to be paid at \$500/LF per the special provision.]
- 2.4 New pile is placed in the hole, backfilled, and driven to refusal at tip elevation. [Driven pile is paid for by the item for Pile Extraction and Redrive.]
- 3.1 A production pile is driven but hangs up above tip elevation. [Paid in accordance with Publication 408, Section 1005.]
- 3.2 It is extracted. [Paid for by the item for Pile Extraction and Redrive.]
- 3.3 The hole is drilled to an elevation above the predetermined pile tip elevation. [Soil drilling is to be paid at \$100/LF and obstruction drilling is to be paid at \$500/LF per the special provision.]
- 3.4 Original pile is placed in the hole, backfilled, but does not reach drill hole elevation. It is extracted and discarded. [No payment is made for the extraction and discarded pile.]
- 3.5 The hole is redrilled to the original drill hole elevation. [No payment is made for redrilling.]
- 3.6 A new pile is placed in the hole, backfilled and driven to refusal at tip elevation. [Redriven pile is paid for by the item for Pile Extraction and Redrive to the drill hole elevation. Driven pile is paid in accordance with Publication 408, Section 1005 for the pile length from

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bottom of drill hole to pile tip elevation.]

- 4.1 A production pile is driven but hangs up above the tip elevation. [Paid in accordance with Publication 408, Section 1005.]
- 4.2 It is extracted. [Paid for by the item for Pile Extraction and Redrive.]
- 4.3 The hole is drilled to the predetermined pile tip elevation. [Soil drilling is to be paid at \$100/LF and obstruction drilling is to be paid at \$500/LF per the special provision.]
- 4.4 The hole is filled with aggregate (less than 20ft.) and then the pile is set up and redriven but does not reach predetermined tip elevation. [No payment is made for the redrive since the contractor elected to backfill the hole prior to redriving the pile.]
- 4.5 The pile is extracted. [No payment is made for the extraction since the contractor elected to backfill the hole prior to redriving the pile.]
- 4.6 The hole is redrilled to the predetermined pile tip elevation. [No payment is made for the redrilling since the contractor elected to backfill the hole prior to redriving the pile.]
- 4.7 The pile is placed in the hole, then backfilled, and driven to tip elevation. [Driven pile is paid for by the item for Pile Extraction and Redrive.]

A pile placed in a predrilled hole is paid for by the item for Pile Extraction and Redrive to the predrilled hole elevation. A pile driven below the predrilled hole elevation is paid in accordance with Publication 408, Section 1005.

10.7.2 Service Limit State Design**10.7.2.2 Tolerable Movements**

The following shall supplement A10.7.2.2.

Design horizontal movements shall not exceed 1/2 in. at the Service Limit State or 1 in. at any Strength or Extreme Limit State (refer to D10.7.2.4).

10.7.2.3 Settlement*10.7.2.3.1 Equivalent Footing Analogy***C10.7.2.2**

The following shall replace AC10.7.2.2.
See AC10.5.2.2 and DC10.5.2.2.

C10.7.2.3.1

The following shall supplement AC10.7.2.3.1.

Settlement predictions for a pile group using an equivalent footing approximation at a depth of $2/3 D_b$ can be very conservative, and do not explicitly account for the effects of length to diameter ratio, L/d , relative pile spacing, s/d , and the number of piles in the group. Elastic solutions have been derived for this problem (Poulos and Davis, 1980; Poulos, 1988) which account for all of these variables. The soil modulus used in these solutions should be chosen, such that it is representative of the general range of the

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10.7.2.4 Horizontal Pile Foundation Movement

The following shall supplement A10.7.2.4.

The provisions of D10.7.3.12.2P and D10.7.3.12.3P shall apply.

C10.7.2.4

The following shall supplement AC10.7.2.4.

The lateral displacement of pile groups may also be estimated using soil-structure methods of analysis such as Reese, et. al., (1987) and Poulos and Davis (1980). The FHWA computer program FB-Pier may also be used.

10.7.3 Strength Limit State Design**10.7.3.2 Point Bearing Piles On Rock***10.7.3.2.1 General**C10.7.3.2.1*

The following shall supplement A10.7.3.2.1.

Certain geologic conditions within the state pose driveability, long-term stability and pile length predictability problems. Such geologic conditions are typically associated with the following characteristics:

- a. Greatly varying depth to bedrock over small areas
- b. Greatly varying quality of bedrock over small areas
- c. Presence of voids, soil-filled seams and/or other discontinuities within bedrock

10.7.3.2.2 Piles Driven to Soft Rock

The following shall supplement A10.7.3.2.2.

If absolute refusal cannot be achieved for a pile bearing in weak rock, the unit bearing resistance shall be estimated by treating the soft rock as soil in accordance with D10.7.3.8.

If the pile can be driven to Case 1 or Case 2 absolute refusal (see Publication 408, Section 1005), the nominal axial resistance shall be estimated in accordance with A10.7.3.2.3.

*10.7.3.2.4P Piles Bearing on Soluble Bedrock**C10.7.3.2.4P*

Where variations in the elevation, quality and discontinuities are present to an appreciable degree in limestone, or other soluble bedrock, the following measures shall be considered:

- a. Use of steel HP 10 x 57 piles designed with axial resistance factor for piles bearing on soluble bedrock as per D6.5.4.2. The piles shall be driven to Case 1 Absolute Refusal (see Publication 408, Section 1005).

Although some limestone formations do not exhibit irregularities and discontinuities to such a degree that pile foundations are adversely affected, comprehensive subsurface exploration, as described in DC10.4.2 is required to evaluate pertinent conditions.

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- b. Tip protection
- c. Allowance for variations in pile location of a minimum of 6 in.
- d. Allowance for deviation from plumbness greater than normally specified
- e. Design of pile caps to withstand loss of up to 35% of piles in a pile group, considered as a localized loss
- f. Use of combined footings where feasible
- g. Relative feasibility and cost of using drilled shaft or micropile foundations in lieu of driven piles

10.7.3.2.5P Piles Bearing on Sloping Bedrock

When piles are driven to end or point bearing on sloping bedrock, special pile tips and driving procedures are appropriate to minimize deflection of piles along the bedrock surface.

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C10.7.3.2.5P

The severity of installation problems increases with increasing hardness of the bedrock surface and decreasing strength of laterally supporting soils above bedrock. These conditions often occur in karst areas. A special driving procedure has been used successfully to promote secure seating.

Piles which are anticipated to bear on sloping bedrock and to experience driving difficulties are to be clearly identified on the bridge plans. A note is to be included on the plans indicating that the specified piles are to be driven in accordance with the procedure delineated below. A special provision is to be included to present the special driving procedure.

The driving procedure for piles seated on sloping bedrock is as follows: Stop driving the pile when the pile tip is at or slightly above bedrock. Continue driving with the stroke of the hammer reduced to 6 inches or to the minimum practical value. For Air or Steam hammers reduce the stroke by reducing the pressure. For Diesel hammers, reduce the stroke by shutting off the fuel and operating the hammer as a drop hammer. When the penetration for 10 to 20 blows is zero, double the stroke and continue driving until the pile is properly seated, or until the maximum energy is obtained from air or steam hammers, or until the stroke becomes limited by the trip device for diesel hammers. Drive the pile to absolute refusal, following the specified driving procedure, unless otherwise directed by the Engineer.

The Engineer may order additional piles to be driven if driving records suggest that any of the piles are not properly seated. The number of piles which are to be properly seated on bedrock must conform to the number shown on the drawings. Payment for the seating of test and bearing piles on sloping bedrock will be in accordance with Sections 1005.4(a) and 1005.4(b) of Publication 408 regardless of any additional piles ordered driven so that the specified

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10.7.3.8 Determination of Nominal Bearing Resistance for Piles*10.7.3.8.1 General*

The following shall supplement A10.7.3.8.1.

For piles bearing on rock, determine the nominal resistance in accordance with A10.7.3.2 and D10.7.3.2.

In general, the Department prefers the use of the semi-empirical methods (α -method, β -Method, λ -Method, Nordlund/Thurman Method) in A10.7.3.8.6 to estimate the axial resistance of piles in soil.

The nominal axial resistance of a pile shall be determined in accordance with the following procedure.

1. Performance and evaluation of a subsurface exploration.
2. Static analysis of pile capacity using the procedures in A10.7.3.2.3 and A10.7.3.8.6, and delivery of the Foundation Submission to the Department for review, in accordance with PP1.9.4.
3. Review of the Foundation Submission by the Chief Bridge Engineer. The Foundation Approval shall stipulate requirements for pile type, estimated length, bearing resistance and stratum, test pile requirements and driving criteria as per PP1.7.5.
4. Based on the proposed pile hammer system, the Department shall perform a wave equation analysis and provide a Pile Hammer Approval.
5. Based on the monitored performance of test piles driven to absolute refusal in accordance with PP1.7.5 and Publication 408, Section 1005, and other testing (i.e., pile load test and/or dynamic monitoring) and analyses (i.e., wave equation analysis), the Department may revise the Foundation Approval and/or the Pile Hammer Approval.

*10.7.3.8.3 Dynamic Testing**10.7.3.8.3aP General*

For high-capacity major structures, erratic bearing strata, or unusual foundation conditions, dynamic testing shall be conducted in accordance with Publication 408,

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number of seated piles matches the drawings. Piles driven in conformance with the above requirements, but determined by the Engineer to be inadequately seated, will be considered acceptable for payment.

C10.7.3.8.1

The following shall supplement AC10.7.3.8.1.

In the case of steel H-piles, the structural capacity of the pile will usually control the design for piles bearing on rock.

The bearing resistance of a pile in soil is derived from the tip resistance and/or shaft resistance, i.e., skin friction. Both the tip and shaft resistances develop in response to foundation displacement. The maximum values of each are unlikely to occur at the same displacement. The shaft resistance is typically fully mobilized at displacements of about 0.1 to 0.4 in.. The tip capacity, however, is mobilized after the pile settles about 8 percent of its diameter (*Kulhawy et al. 1983*).

C10.7.3.8.3aP

Dynamic testing entails measurement and evaluation of force and acceleration of the pile during driving to estimate pile axial resistance, hammer energy, driving stresses and

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Section 1005, where economically feasible for any or all of the following:

- a. To provide estimates of nominal axial pile resistance under actual field conditions
- b. To provide estimates of pile freeze and pile relaxation through redrive testing of previously monitored piles, and/or comparison with static-load test results
- c. To provide, in conjunction with CAPWAP analysis, accurate quake and damping factors for use in refined wave equation analyses and/or further dynamic monitoring
- d. To measure hammer energy transmitted to the pile, to permit evaluation of driving stresses, pile damage and other driveability factors

Dynamic testing may be performed in conjunction with test piles and static load testing of selected test piles to verify resistance estimates or provide a basis for adjusting wave equation input data. Dynamic testing is particularly applicable for piles deriving an appreciable portion of their resistance from end bearing on rock or for piles driven to blow counts exceeding ten blows per 1 in.

10.7.3.8.3bP *Input Factors*

Pile property and geologic input factors for dynamic testing shall accurately reflect pile, soil and rock properties. Input factors for pile properties shall be obtained from pile manufacturer literature. Preliminary soil and rock input factors (i.e., damping and quake) can be obtained from Table D10.7.3.8.4dP-1. Refined soil and rock input factors resulting from CAPWAP analysis shall be used as they become available.

10.7.3.8.3cP *Correlation with Static Load Tests*

When dynamic testing is performed in conjunction with static load testing, the strengths obtained by both wave equation analysis and dynamic testing shall be calibrated with the load test results as described in A10.7.3.8.2. Correlation shall consist of tabulating the difference between static load test strengths, wave equation analysis, and dynamic monitoring strength estimates, and assessing the consistency of overprediction or underprediction of static test nominal strengths by dynamic methods. If redriving is performed to evaluate time-dependent strength

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other related parameters. Manipulation of the measured quantities is performed rapidly in the field using a portable minicomputer, which applies the measured input data to the solution of a dynamic equilibrium analysis of the pile. Detailed discussion of dynamic data acquisition and analysis is presented in the Dynamic Monitoring Manual.

The dynamic testing equipment presently used estimates the mobilized soil resistance and equates it to the nominal pile resistance. When pile sets are equal to or less than the quake value of the bearing strata, dynamic estimates of pile capacity tend to be conservative because full soil resistance is not mobilized. While conservative strength estimates are typical at small sets under normal conditions, dynamic capacity estimates may be unconservative where relaxation or bearing materials with high damping are present. This emphasizes the importance of redrive testing (D10.7.3.8.5) as a means of adjusting initial dynamic strength estimates to allow for unusual conditions.

C10.7.3.8.3bP

The greatest source of inaccuracy in dynamic testing input is typically the quake and damping factor input. To provide the most accurate values of quake and damping for use in dynamic testing, CAPWAP analyses should be performed as soon as dynamic data acquisition begins. CAPWAP analyses involve digitalization of the measured force and velocity traces and subsequent modeling of the measured force trace by iterative application of progressively refined values of quake, damping, and load distribution to match the measured velocity trace. Detailed discussion of CAPWAP analyses is presented in the Dynamic Monitoring Manual.

C10.7.3.8.3cP

A correlation between dynamic capacity estimates and static load test capacities of 10% (where correlation % = [(Dynamic Test Capacity - Static Load Test Capacity) / Static Load Test Capacity] x 100) is considered accurate. When this correlation is achieved, an abbreviated static load testing program may be applied to those remaining portions of the project involving similar soil and rock conditions.

When pile driving criteria involve final sets of less than 0.05 to 0.1 in. per blow (i.e., driving resistances greater than 10 to 20 blows per in.) load testing should be performed in

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changes, the dynamic monitoring strength obtained at the beginning of redriving shall also be correlated with static load test results. The designer shall evaluate the consistency of dynamic monitoring strength estimates with regard to their reliable use for strength determination after initial static load testing has been completed. If no acceptable consistent correlation is determined, continued dynamic monitoring during the remainder of pile installation shall serve the primary purpose of verifying of acceptable hammer performance and driving stresses.

10.7.3.8.3dP Redriving

Pile freeze or relaxation can be determined by redrive testing of previously installed piles. A minimum of five days shall elapse between the end of initial driving and redrive testing, unless otherwise approved by the Chief Bridge Engineer. Care shall be taken to minimize hammer energy fluctuations because at small pile sets, differences in hammer energy between the end of initial driving and the beginning of redrive testing will affect dynamic capacity estimates.

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conjunction with dynamic monitoring.

C10.7.3.8.3dP

Generally, the factors most affecting the validity of redriving results (with respect to real pile strength changes which have occurred) relate to differences in hammer performance between the end of initial driving and the start of redriving. Fluctuations in hammer energy between initial driving and redriving complicate the comparison of driving resistances and, in most cases, dynamically monitored strengths. Driving resistances observed at one hammer energy may not easily be compared to driving resistances obtained at other energies. Therefore, it is often difficult to draw meaningful conclusions regarding pile freeze or relaxation when hammer energies vary. In the case of dynamic capacity estimates at relatively small pile sets per blow (generally sets less than the quake value of the soil or rock supporting the pile), the full capacity of the pile is not mobilized and the capacity determined by dynamic monitoring tends to reflect the hammer energy expended on the pile rather than the actual pile capacity. Thus, when piles are driven or redriven at relatively high driving resistances, hammer energy fluctuations may be reflected as pile strength changes which have not actually occurred.

The problems described above illustrate the importance of consistent hammer performance for evaluation of redrive testing. Most hammers (e.g., diesel, air-steam) have certain operating characteristics which tend to introduce some degree of variation between the end of initial driving performance and the beginning of redriving performance. Double-acting hammers generally perform at optimum levels during moderately hard driving (i.e., 5 to 15 blows per 1 in.), but may experience cylinder lift at higher driving resistances which necessitates some throttle reduction. Most double-acting hammers must be raised up to full throttle in several steps; thus, the first several hammer blows at the start of redriving usually occur at much less than optimum energy. Single-acting diesel hammers may experience some loss of transmitted energy during hard driving due to increased frictional losses related to heating of the hammer. All of these hammer performance characteristics may influence the execution and/or interpretation of redrives, depending upon the differences in driving resistances and hammer condition between the end of initial driving and redriving.

The different hammer types may be approximately

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10.7.3.8.3eP Capacity Prediction

Nominal axial pile resistances shall be estimated for monitored piles throughout the range of driving resistances appropriate for the type of pile(s) and support mechanism(s). Resistance estimates based on dynamic monitoring shall be representative of an average value of at least five hammer blows.

*10.7.3.8.4 Wave Equation Analysis**10.7.3.8.4aP General*

The constructability of the pile foundation design shall be evaluated using GRLWEAP 2003. Wave equation analyses shall be performed on all pile/hammer/soil/rock combinations pertaining to the foundation design. The wave equation shall be used to confirm that the design pile section can be installed to the designed depth, nominal axial resistance, and within the driving stress levels specified in D10.7.8 using the resistance factors specified or referred to in Table D10.5.5.2.3-1. The wave equation analyses will be performed by the Department based on hammer data submitted by the contractor. Revised wave equation analyses incorporating actual field conditions (e.g., as determined by pile installation, static load tests, dynamic monitoring, or CAPWAP analyses) shall be performed during the design phase if directed by the Department. Each analysis shall include a completed "Hammer Approval" form. Analyses shall reflect optimized driveability as set forth in D10.7.8.

10.7.3.8.4bP Pile Driving Systems

Wave equation analyses shall incorporate appropriate hammer performance parameters which are furnished by the contractor when properly completed pile hammer data sheets are submitted. For hammers with variable throttle settings and strokes, the settings and strokes to be used shall be identified. Cushion, capblock and drive helmet materials, and the associated weights and stiffness and restitution coefficients shall be identified.

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ranked with respect to the amount of performance variation and resultant difficulty in interpreting redrive data. Ranked from least to most difficult, the following sequence is indicated: single-acting air/steam, single-acting diesel, double-acting air/steam and, double-acting diesel. The designer should recognize the potential difficulties in using a particular hammer type for redriving operations and, where appropriate, should restrict hammer types to those which will minimize problems in obtaining usable information.

C10.7.3.8.3eP

Use of an average capacity value over a range of at least five blows per inch is suggested to minimize the effect of the variations in hammer performance and capacity estimation, which typically occur between hammer blows during driving.

C10.7.3.8.4aP

Detailed discussion of the wave equation analysis program background and use are presented in the various user manuals currently available, Goble and Rausche (1987), and the Department's Wave Equation Analysis Manual. Publication 15A provides a compilation of wave equation analyses conducted by the Department in conjunction with previous work.

Wave equation analyses generally predict driving stresses within about 20% low to 10% high, Gannett Fleming, et al, (1985) and Thompson and Thompson (1983). Static pile strengths are usually predicted with more varying degrees of accuracy, depending upon the reliability of quake and damping factors and the load carrying mechanism assumed in the analysis. The wave equation analyses are usually more useful in estimating pile driving stresses than in estimating nominal axial resistance.

C10.7.3.8.4bP

Where dynamic monitoring or field observations indicate appreciable differences between driving system parameters used in wave equation analyses and those actually occurring in the field, the analyses should be rerun using the revised input, representative of field conditions, or operating procedures should be made to achieve the appropriate relationship between driving resistance and pile capacity.

The values of hammer efficiencies used in the current

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10.7.3.8.4cP Pile Characteristics

The analyzed pile shall be modeled with respect to the anticipated soil and rock strata penetrated, length of pile above and below ground, and cross-sectional variations. Input data shall accurately reflect the presence of splices, tip protection, soil plugs (in open-end pipe piles) and similar features. The properties and dimensions of the mandrel shall be used for analysis of mandrel-driven piles.

10.7.3.8.4dP Quake and Damping Factors

Wave equation analyses shall incorporate quake and damping factors appropriate to the anticipated character and relative position of the soil and rock strata penetrated. Quake and damping factors suitable for preliminary analyses are given in Tables 10.7.3.8.4dP-1 and 10.7.3.8.4dP-2. If available, values of quake and damping factors based on dynamic monitoring and/or CAPWAP analysis shall be used.

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wave equation programs generally reflect optimum hammer performance, whereas, actual hammer performance is typically less efficient. Therefore, the magnitude of transmitted hammer energy predicted by wave equation analyses typically exceeds that observed in the field. The lower efficiency results in lower driving stresses compared to those predicted by wave equation analyses.

C10.7.3.8.4dP

Quake and damping factor(s) are used in wave equation analyses to model the elasto-plastic behavior and dynamic impedance of the soil and rock profile through and into which the pile is driven. Variations in the values used in analyses, particularly for damping factors, may exert appreciable influence on the static pile strength predicted as a function of driving resistance. The higher the values of quake and damping factor used in the analysis the lower the capacity predicted at a particular driving resistance. In cases where the pile is driven through layers of greatly differing damping characteristics (e.g., granular and cohesive layers, or granular material overlying soft rock), effects of soil layering should be considered in the prediction of pile performance.

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Table 10.7.3.8.4dP-1 – Preliminary Quake Factors for Impact Driven Piles (Pile Dynamics, Inc. 2003).

SOIL TYPE	PILE TYPE OR SIZE	SKIN QUAKE***	TOE QUAKE
All soil types	Non-displacement piles* i.e. driving unplugged	0.10 in	0.10 in
Very dense or hard soils	Displacement piles** of diameter or width D	0.10 in	D/120
Loose or soft soils	Displacement piles** of diameter or width D	0.10 in	D/60
Soft Rock	Non-displacement piles* i.e. driving unplugged	0.10 in	0.10 in
Hard Rock	All types	0.10 in	0.05 in

* Non-displacement piles are sheet piles, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches or less will plug during driving into a bearing layer. Between 20 to 30 inches, pipe piles may or may not plug.

** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, H-Piles and pipe piles with diameters 20 inches or less would be modeled as displacement piles.

*** For piles which require a residual stress analysis this value shall be 0.05 in.

Table 10.7.3.8.4dP-2 – Preliminary Damping Factors for Impact Driven Piles (Pile Dynamics, Inc. 2003).

SOIL TYPE	SKIN DAMPING*	TOE DAMPING
Non-cohesive soils	0.05 sec/ft	0.15 sec/ft
Cohesive soils	0.20 sec/ft	0.15 sec/ft
Rock (End or Point Bearing Pile)	0.05 sec/ft	0.10 sec/ft

* For mixed soils, intermediate values may be appropriate.

10.7.3.8.4eP Nominal Axial Resistance

The nominal axial pile resistance can be predicted from the relationship between driving resistance and nominal axial resistance developed from the wave equation analysis. Wave equation analyses used for nominal axial resistance prediction shall meet the optimized driveability requirements of D10.7.8 at the driving resistances specified for the various pile support mechanisms (point bearing, end bearing or friction).

10.7.3.8.5 Dynamic Formula

The following shall replace A 10.7.3.8.5.

A Dynamic Formula should not be used to establish the driving criterion unless pre-approved by the Chief Bridge Engineer.

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10.7.3.8.6 *Static Analysis*10.7.3.8.6a *General*

C10.7.3.8.6a

The following shall supplement AC10.7.3.8.6a.

The nominal axial resistance of piles in cohesive soils may be calculated using a total stress method (e.g., Tomlinson (1957)) for undrained loading conditions, or an effective stress method (e.g., Meyerhof (1976)), for drained loading conditions. The nominal axial resistance may also be calculated from in situ testing methods, such as the cone penetration (e.g., Schmertmann (1978)).

The nominal axial resistance of piles in cohesionless soils may be calculated using an empirical effective stress method (e.g., Nordlund) or from in-situ testing methods and analysis, such as the cone penetration (e.g., Schmertmann (1978)).

10.7.3.8.6c *β -Method*

C10.7.3.8.6c

The following shall supplement AC10.7.3.8.6c.

The term β implicitly includes the effects of variations in interface angle of shearing resistance and the in-situ horizontal stress state, and may be taken as:

$$\beta = k \tan \delta \quad (\text{C10.7.3.8.6c-1P})$$

where:

k = post-construction lateral earth pressure coefficient (dim)

δ = effective stress angle of shearing resistance between soil and pile (deg)

10.7.3.8.6e *Tip Resistance in Cohesive Soils*

C10.7.3.8.6eP

Equation A10.7.3.8.6e-1 is a simplified expression for the nominal unit tip resistance of a pile for undrained loading which is applicable only when the following criteria are met:

- the weight of the pile is approximately equal to the weight of soil it displaces,
- $D_f/D \geq 5$, and
- $E_s/3S_u \geq 8$.

10.7.3.8.6f *Nordlund/Thurman Method in Cohesionless Soils*

The following shall supplement A10.7.3.8.6f.

The Department's preferred method for predicting side resistance of piles in cohesionless soils is the Nordlund/Thurman Method.

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*10.7.3.8.6g Using SPT or CPT in Cohesionless Soils**C10.7.3.8.6g*

The following shall supplement AC10.7.3.8.6g.
CPT is typically only used in fine sands and silts. It is not practical for coarse sands and gravels.

10.7.3.9 Resistance of Pile Groups in Compression**C10.7.3.9**

The following shall supplement A10.7.3.9.

The resistance factor for an equivalent pier or block failure shall be as given in Table D10.5.5.2.3-1. The resistance factors for the group resistance calculated using the sum of the individual resistances are the same as those for the single pile resistance as given in Table D10.5.5.2.3-1.

The following shall supplement AC10.7.3.9.

The additional resistance of the cap shall be ignored.

The efficiency of pile groups in cohesive soil may be diminished from the individual pile due to overlapping zones of shear deformation in the soil surrounding the piles.

10.7.3.10 Uplift Resistance of Single Piles

The following shall supplement A10.7.3.10.

Uplift resistance at the Service Limit State shall not be used without approval by the Chief Bridge Engineer.

10.7.3.11 Uplift Resistance of Pile Groups

The following shall supplement A10.7.3.11.

Uplift resistance shall not be used without approval by the Chief Bridge Engineer.

10.7.3.12 Nominal Lateral Resistance of Pile Foundations*10.7.3.12.1P Batter Piles**C10.7.3.12.1P*

The batter piles in a pile group shall resist the entire lateral load through the lateral component of their axial capacity, unless an alternate method is approved by the Chief Bridge Engineer.

Refer to D6.15.1 for design procedures for pile groups containing battered piles.

This method is used in the Department's computer programs.

*10.7.3.12.2P Vertical Piles**C10.7.3.12.2P*

The nominal resistance of pile foundations to lateral loads shall be evaluated based on both geomaterial and structural properties. The lateral soil resistance along the piles should be modeled using *P-y* curves developed for the soils at the site.

The applied loads shall be factored loads and they must include both lateral and axial loads. The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group. Minimum embedment of piles into the pile footing shall be in accordance with D10.7.1.2 and Standard Drawing BD-621M.

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For this embedment, the piles shall be designed assuming both full pile-head fixity and 50% pile-head fixity. The 50% pile-head fixity condition shall be simulated by application of one-half of the fixed-head moment (as a negative moment) to the top of a free-head pile.

The passive resistance of soil in front of the footing shall be neglected.

Contractors shall be required to replace any disturbed soil or fill voids created during driving of the pile with compacted granular material.

The final design of laterally loaded vertical piles (i.e., no batter piles in the pile pattern) shall be based on the results of COM624P computer analyses (see Wang and Reese, (1993); and Reese, (1984)), LPILE 5.0 (see ENSOFT, Inc. 2004 for LPILE 5.0), or other methods of analysis (e.g., Borden and Gabr (1987), for the case of a sloping ground surface), if approved by the Chief Bridge Engineer, which account for the effects of soil/rock-structure interaction between the pile and ground. Other methods of analysis to evaluate the nominal horizontal resistance or deflection of laterally loaded piles may be used for preliminary design only as a means to determine approximate shaft dimensions.

Design of axially- and laterally-loaded piles using COM624P, Wang and Reese (1993), or LPILE 5.0 shall be performed according to the following steps (for integral abutments, refer to Appendix G):

1. Select a preliminary pile section based on settlement and axial load considerations (D10.7.2.3 and D10.7.3).
2. Using factored axial and lateral loads, compute the maximum groundline lateral deflection and the maximum factored moment in the pile section using COM624P or LPILE 5.0.
3. If the groundline lateral deflection exceeds 1 in. or the maximum factored moment exceeds the factored moment resistance of the pile obtained from D5.13.4 and A5.13.4, and D6.15, select a new trial section, and repeat Step 2.
4. If neither the groundline lateral deflection nor the factored moment criteria in Step 3 is exceeded, compute the maximum groundline lateral deflection of the pile for the service limit state using COM624P or LPILE 5.0.
5. If the groundline lateral deflection exceeds 1/2 in., select a new trial section and repeat Step 2.

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The negative moment used to model partial pile-head fixity acts in a direction opposite that of the actual moment.

The degree of pile-head fixity is a function primarily of the pile-head embedment into the pile footing. Based on analytical studies and full-scale load test results, an embedment length of two to four pile diameters is required to provide full fixity, Shahawy and Issa, (1992); Castilla, et al, (1984).

The lateral loading of pile groups containing vertical and/or batter piles may be analyzed using soil structure methods of analysis, such as Reese, et al, (1994) and Poulos and Davis (1980).

When this analysis is performed, the loads are factored since the strength limit state is under consideration, but the resistances as represented by the *P-y* curves are not factored since they already represent the ultimate condition.

COM624P or LPILE 5.0 requires the engineer to make a decision as to whether the applied loading is static or cyclic in nature. This decision affects the *P-y* curves generated internally by the program in determining the level of soil resistance. The following guideline is suggested for typical applications encountered on Department projects:

When the predominate lateral load on the pile/caisson is sustained and non-transient in nature (e.g. earth pressure on retaining walls, wingwalls and abutments), use static soil moduli and static loading condition;

When the predominate lateral load on the pile/caisson is transient in nature (e.g. wind, centrifugal and braking loads on piers), use cyclic soil moduli and cyclic loading condition.

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For all COM624P or LPILE 5.0 analyses, lateral load deflection relationships used to determine deflections should be unfactored, whether input or default relationships are used.

The minimum penetration of the piles below ground (see D10.7.6) required in the contract should be established such that fixity is obtained. For this determination, the loads applied to the pile are factored as specified in Sections A3 and D3, and a soil resistance factor of 1.0 shall be used as specified in Table D10.5.5.2.3-1.

If fixity cannot be obtained, additional piles should be added, larger diameter piles used if feasible to drive them to the required depth, or a wider spacing of piles in the group should be considered to provide the necessary lateral resistance. Batter piles may be added to provide the lateral resistance needed, unless downdrag is anticipated. If downdrag is anticipated, batter piles should not be used. The design procedure, if fixity cannot be obtained, should take into consideration the lack of fixity of the pile.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D 3966.

10.7.3.12.3P Group Lateral Load Resistance

Laterally loaded pile groups containing battered piles shall be designed assuming that all lateral load is resisted by the horizontal component of the axial resistance of the battered piles. Laterally loaded pile groups containing only vertical piles shall be designed based on lateral load analysis of individual vertical piles in accordance with A10.7.3.8.2, assuming that the lateral load on the pile group is evenly distributed among the piles. The distribution of vertical loads within both mixed and all vertical pile groups shall be determined based on elastic theory in accordance with the Department's computer programs.

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The strength limit state for lateral resistance is only structural (see Sections A5, A6, D5 and D6 for structural limit state design requirements), though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state, and this limit state is reached, in the general case, when the nominal combined bending and axial resistance is reached.

For information on analysis and interpretation of load tests, see A10.7.2.4 and D10.7.2.4.

Refer to Publication 15A, Compilation of Pile Load Test and Wave Equation Analysis (1989).

C10.7.3.12.3P

Driven piles in a group are considered to act individually when the center-to-center spacing is greater than 3D in the direction perpendicular to the applied lateral load, and greater than or equal to 8D parallel to the direction of loading. For the case of closely-spaced piles in a group, the interaction behavior is typically accounted for indirectly, using empirical procedures (e.g., Reese, et al, (1994) and Prakash and Sharma (1990)). One such procedure assumes a reduction in the coefficient of lateral subgrade reaction for a pile in a group from that of a single pile, using the following ratios, Prakash and Sharma (1990):

Table C10.7.3.12.3P-1 - Ratio of Lateral Resistance of Pile in Group to Lateral Resistance of Single Pile

Center-to-Center Pile Spacing [parallel to direction of applied load]	Ratio of Lateral Resistance of Pile in Group to Single Pile	
	Cohesive Soil	Cohesionless Soil
8D	1.00	1.00
6D	0.65	0.70
4D	0.50	0.60
3D	0.40	0.50

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For typical Department designs, in which the center-to-center spacing is 3D or greater in the direction parallel to the loading, and the lateral displacement is limited to 1 inch at the strength limit state, the factors in Table C10.7.3.12.3P-1 need not be applied because there is no significant stress overlap between adjacent piles for this condition. Reductions of the magnitudes indicated in Table C10.7.3.12.3P-1 are consistent with displacements at passive soil failure (i.e., about 3 in. to 5 in.).

10.7.3.13 Pile Structural Resistance*10.7.3.13.4 Buckling and Lateral Stability*

The following shall supplement A10.7.3.13.4.

For a pile group composed of only vertical piles, which is subject to lateral loads, the pile structural analysis shall include explicit consideration of soil-structure interaction using a COM624P or LPILE 5.0 analysis, as specified in D10.7.3.12.

10.7.3.13.5P Point Attachments on Steel H-Piles

If pile penetration through cobbles, boulders, debris fill or obstructions is anticipated and for all piles driven to bedrock, pile tips shall be reinforced with structural shapes or with prefabricated cast steel points. Cast steel points shall meet the requirements of ASTM A 27.

See Standard Drawing BC-757M and Publication 408, Section 1005, for details.

10.7.5 Corrosion and Deterioration**10.7.5.1P General**

Evaluation of protective measures shall include consideration of the soil and groundwater conditions surrounding the pile and the loading characteristics of the pile. The evaluation of protective measures shall be performed for each situation based on the level of deterioration anticipated, the practicality of applying a particular protective measure and cost.

See D10.7.5.6P for the definition of conditions indicative of potentially corrosive soil and groundwater.

C10.7.5.1P

Soil and groundwater characteristics, such as pH, resistivity, sulfate content, chloride content and bacteria level are necessary for determining the deterioration potential of a pile.

Methods of pile deterioration prevention should be compared for practicality and cost. For a small job in which piles are driven through a layer of mildly corrosive soil, the deduction of 1/16 in. in determining the pile capacity may be practical and the cost of excess material in the non corrosive area insignificant. On a large job, applying a protective coating to only that section of the pile exposed to the corrosive layer may result in cost savings. In some cases, the choice of pile type (i.e., steel vs. concrete) may be made on the basis of deterioration considerations.

High velocity water flows containing suspended sediments can abrade piles and remove protective coatings above the scour depth.

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10.7.5.2P Steel Piles

The following measures shall be compared for protection of steel piles against deterioration by corrosion.

- a. Deduct 1/16 in. (minimum) from the exposed surface of the pile used to compute section capacity if anticipated corrosion losses will be less than 1/16 in.
- b. Apply a coating which has good dielectric strength, is resistant to abrasive forces during driving, and has a proven service in the type of corrosive environment anticipated. Electrostatically applied epoxies have proven to be effective in many cases.
- c. Place a minimum 4 in. thick concrete encasement jacket around the pile of the same quality concrete as that recommended for concrete piles.
The portion of the steel embedded in the bottom few feet of the concrete jacket shall be coated with an electrostatically applied epoxy.
- d. Install a cathodic protection system, and coat the pile with a coating resistant to cathodic disbondment to reduce cost of the system. Piles exposed to unpolluted fresh water shall be electrically connected so that a cathodic protection system can be easily installed if corrosion is discovered during future inspections.

10.7.5.3P Concrete Piles

In any corrosive medium, a dense, impervious concrete shall be used. The following measures shall be taken on all concrete piles used in corrosive environments:

COMMENTARY

C10.7.5.2P

A steel pile foundation design should consider that steel piles may be subject to corrosion, particularly in fill soils, low pH soils (acidic) and marine environments. A field electric resistivity survey, or resistivity testing and pH testing of soil and groundwater samples should be used to evaluate the corrosion potential.

Corrosion rates for piles in undisturbed soil are generally negligible, Schwerdtfeger and Romanoff (1972). In fills, the corrosion rates range from negligible to severe (averages range from 2 to 8 mils per year) depending on the various factors. Corrosion usually takes place in the form of pitting which is not as serious as a uniform reduction in thickness over a considerable area of the pile. The rate of corrosion slows up considerably as the steel takes on a film of corrosion products which tends to protect the steel from further corrosion. Where steel piles are driven into sand, conditions are particularly favorable to the formation of an impervious, insoluble coating of ferrosilicate as soon as the steel corrodes slightly, U. S. Steel (1986).

Steel piles in fresh water have a slow initial corrosion rate of 1 to 3 mils per year, decreasing with time as a protective coating forms, U. S. Steel (1986). The corrosion rate of steel piles in polluted water is highly variable.

Although coal-tar epoxies are commonly used on piles, the Department has experienced some problems with their use. Plain epoxy coatings have proven to be more successful. Other coatings, such as metalized zinc and aluminum with top coats, are usually more expensive, but may be applicable in some cases. Coatings, such as vinyls, epoxies, urethanes and coal tar epoxies have been found to be effective in controlling corrosion in water.

Cathodic protection systems are very expensive due to required maintenance. These systems, in combination with coatings, are used in buried pipeline applications where the effects of small corrosion pits at defects in the coating can lead to leaks. In piles, however, small pitting is not as critical, and a good coating without cathodic protection is generally effective in preventing significant uniform corrosion loss on the pile section.

C10.7.5.3P

A concrete pile foundation design should consider that deterioration of concrete piles can occur due to sulfates in soil, groundwater or sea water, chlorides in soils and chemical wastes, acidic groundwater and organic acids. Laboratory testing of soil and groundwater samples for

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- Minimum concrete cover as follows:
 - Cast-in-place reinforced concrete, 4 in.
 - Precast reinforced concrete, 3 in.
 - Prestressed concrete - Prestressed strands, 2 1/2 in.; secondary reinforcement, 1 1/2 in.
- Maximum water/cement ratio of 0.45 (by weight)
- Air entrainment
- No concrete additives containing chlorides

The following protective measures may also be required in particular cases.

Sulfate resistant cement as follows:

Water Soluble Sulfate in Soil (%)	Sulfate in Water (ppm)	Cement Type
0.10 - 0.20	150 - 1,500	II
0.20 - 2.00	1,500 - 10,000	V
> 2.00	> 10,000	V plus Pozzolan

- Epoxy-coated reinforcement
- Cathodic protection with electrical continuity between all reinforcement. Cathodic protection should not be used for prestressed piles.

10.7.5.4P Timber Piles

Untreated timber piles shall be used only for temporary construction. Timber piles for permanent construction shall be protected as follows:

- Treatment with creosote in accordance with Publication 408, Section 1005, or
- Concrete jacketing, in accordance with recommendations for concrete piles

10.7.5.5P Preliminary Testing

If driven pile foundations are anticipated, the soils investigation shall provide the following minimum information to determine pile deterioration potential:

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sulfates and ph is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and groundwater samples is recommended when chemical wastes are suspected.

Sulfates are present in groundwater in fills containing blast furnace slag, cinders, or pyritic shale. Sulfates react with chemicals present in the concrete, such as hydrated lime and gypsum. These reactions result in an increase in solid volume, with a subsequent spalling of concrete.

The principle cause of reinforcing steel corrosion is chloride ion. Good corrosion protection in concrete piles can be accomplished by limiting the amount of chloride ion in the concrete mix. ACI Committee 222 has suggested the following limits for chloride ion in concrete prior to service exposure, expressed as a percent by weight of cement:

- Prestressed concrete 0.06%
- Reinforced concrete exposed to chloride in service 0.10%
- Reinforced concrete in a moist environment, but not exposed to chloride 0.15%

The use of accelerating admixtures containing calcium chloride is discouraged.

C10.7.5.4P

The principle causes of deterioration of timber piles are decay, insect attack, and marine-borer attack. Deterioration of timber piles typically occurs in areas where oxygen and moisture are present. Therefore, piles driven below the permanent groundwater table are generally unaffected by decay and insect attack. Treatment with creosote is the most common method of protecting timber piles. The advantages of creosote as a preservative are its high toxicity to wood-destroying organisms, relative insolubility, availability, good depth of penetration and good performance record.

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- Soil pH, sulfate content in soil and groundwater and moisture content
- General soil profile, including type, variation, depth and layering of fill and undisturbed natural soils, and groundwater level
- Previous land use
- Soil resistivity (laboratory test on soil samples) - If evaluation of data with respect to criteria in D10.7.5.6P indicates a potential corrosion problem, a field resistivity survey may be performed.

If piles extend through open water, the water shall be tested for chlorides, sulfates, bacteria and pH, and its velocity shall be measured.

10.7.5.6P Corrosive Environments

Conditions which are indicative of potentially corrosive soil and groundwater and require consideration of protective measures:

- Resistivity less than 2000 ohm-cm in soil
- Resistivity between 2000 and 5000 ohm-cm and combined with:
 - sulfate concentration greater than 200 ppm, or
 - chloride concentration greater than 100 ppm
- pH less than 5.5
- pH between 5.5 and 8.5 in soils with high organic content
- Sulfate concentration greater than 1000 ppm in soil or greater than 150 ppm in groundwater
- Landfills and cinder fills
- Soils subject to mine or industrial drainage
- Mixtures of high resistivity soils and low resistivity high-alkaline soils

Conditions which are low in corrosion potential and which generally do not require protective measures include:

- Undisturbed natural soils with no free draining layers, regardless of conditions, noted above as indications of high corrosion potential
- pH greater than 5.5 with no organic content

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- Soils with resistivities greater than 5000 ohm-cm and uniform in profile
- Well-aerated loose soils of uniform composition (i.e., sand)

Water shall be considered corrosive if it contains any of the following:

- Chloride content greater than 1000 ppm
- Sulfate content greater than 150 ppm
- Mine or industrial runoff
- High organic content
- pH less than 5.5

Water with high velocity is generally more damaging than standing water.

Piles exposed to air containing sulfur dioxide, chlorine concentrations, or other pollutants require protection against deterioration.

10.7.5.7P Stray Currents

Steel and concrete piles located near sources of direct currents (i.e., electric transit systems, welding shops, cathodic protection systems) may be subject to damage from stray currents. To protect against stray current damage, steel piles shall be electrically connected and grounded to the current source. Concrete piles shall be similarly grounded with electrical continuity between all reinforcement. The effects of stray currents on prestressed piles can lead to pile failure and prestressed piles should not be used in areas of potential stray currents.

10.7.6 Determination of Minimum Pile Penetration**C10.7.6**

The following shall supplement A10.7.6.

In general, unless refusal is encountered, the design penetration for any pile should be not less than 10.0 ft. into hard cohesive or dense granular material and not less than 20.0 ft. into soft cohesive or loose granular material.

Unless refusal is encountered, piles for trestle or pile bents shall penetrate a distance equal to at least one-third the unsupported length of the pile.

Piling used to penetrate a soft or loose upper stratum overlying a hard or firm stratum, shall penetrate the firm stratum by a distance sufficient to limit movement of the piles and attain sufficient bearing capacities.

Penetration shall be controlled so that pile damage during driving is avoided. A maximum blow count limited by driving stresses shall be determined using a Wave Equation analysis (see D10.7.3.8.4 and DC10.7.3.8.4). The

The following shall supplement AC10.7.6.

Driving points or shoes may be necessary to achieve penetration or to provide adequate lateral tip restraint.

The generally acceptable minimum length of 10 ft. is based upon driveability and lateral resistance concerns. As

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maximum blow count shall not be exceeded.

Although the minimum design length of piles in hard or dense soils is generally 10 ft., reduced penetrations as low as 6 ft. may be permitted for special conditions. The minimum penetration length of pile shall be 10 ft. unless otherwise approved by the Chief Bridge Engineer for a specific project.

For friction and end bearing piles, the designer shall require predrilling through the resisting soil layer to the bottom of the estimated scour depth prior to the pile driving.

10.7.8 Driveability Analysis**10.7.8.1P General**

Driveability shall be optimized using the procedure specified in D10.7.3.8.4 when the design resistance is achieved with the least expensive pile and most efficient hammer sized for maximum set without damaging the pile.

The establishment of the installation criteria for driven piles should include a drivability analysis. Except as specified herein, the drivability analysis shall be performed by the Engineer using a wave equation analysis, and the driving stresses (σ_{dr}) anywhere in the pile determined from the analysis shall be less than the limits contained herein:

The following permissible driving stresses, as determined by wave equation analysis at 20 blows per 1 in., are specified for particular pile types.

- a. Driven Steel Piles Driven without Mandrel
 - $\sigma_{dr} = \phi_{da} f_y$ for $f_y = 36$ ksi
 - $\sigma_{dr} = 0.8\phi_{da} f_y$ for $f_y = 50$ ksi (for point bearing piles provided the piles have verification of material strength greater than or equal to 50 ksi)

where:

f_y = yield strength for steel (ksi)

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pile length decreases, the potential for damage during driving increases, particularly for point bearing and end bearing piles. Because very short piles are nearly always point bearing or end bearing piles, damage potential is a prime consideration. Lateral resistance is severely lessened in short piles due to reduced tip fixity and decreased soil resistance adjacent to the pile.

Because of these concerns, the designer shall conduct a thorough analysis of driveability (A10.7.8 and D10.7.8) and lateral load resistance (D10.7.2.4 and D10.7.3.12) during design to evaluate the feasibility of short piles for a particular application.

Predrilling will reduce side resistance through the scour zone during pile driving so as to reduce the probability of obtaining a false indication of adequate pile capacity.

C10.7.8.1P

Because dynamic resistance to pile penetration must be overcome during driving, but does not influence subsequent static capacity, the highest stress levels experienced by a pile will occur during driving.

Optimized pile design generally results in piles being driven at near-maximum permissible stresses to the driving criteria for the various load carrying mechanisms. Some exceptions include the cases of friction/displacement piles where pile length reductions in conjunction with less than maximum possible loads may provide an economically attractive combination. Such cases are relatively rare, however, and are difficult to anticipate during the design phase of a project without field testing.

Wave equation analyses should be conducted during design using a range of likely hammer/pile combinations, considering the soil and installation conditions at the foundation site. See D10.7.3.8.4 for additional considerations for conducting wave equation analyses. These analyses should be used to assess feasibility of the proposed foundation system and to establish installation criteria with regard to driving stresses to limit driving stresses to acceptable levels. For routine pile installation applications, e.g., smaller diameter, low nominal resistance piles, the development of installation criteria with regard to the limitation of driving stresses, e.g., minimum or maximum ram weight, hammer size, maximum acceptable driving resistance, etc., may be based on local experience, rather than conducting a detailed wave equation analysis that is project specific. Local experience could include previous drivability analysis results and actual pile driving experience that are applicable to the project specific situation at hand. Otherwise, a project specific drivability study should be conducted.

Drivability analyses may also be conducted as part of

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$$\phi_{da} = 1.0$$

- b. Steel Piles Driven with Mandrel - Driving stresses will be controlled by shell damage, dependent on geologic conditions and obstructions
- c. Concrete Piles

Compression:

$$\sigma_{dr} = \phi_{da} (0.85 f'_c - f_{pe}) \quad (10.7.8.1P-1)$$

Tension:

$$\sigma_{dr} = \phi_{da} (0.095 \sqrt{f'_c} + f_{pe}) \quad (10.7.8.1P-2)$$

where:

f'_c = compressive structural design strength of concrete at 28 days (ksi)

f_{pe} = concrete compression stress due to prestressing after all losses (ksi)

For reinforced concrete piles (non-prestressed piles), f_{pe} equals zero.

- d. Timber Piles - See Publication 408, Section 1005

For routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived.

This drivability analysis shall be based on the maximum driving resistance needed:

- To obtain minimum penetration requirements specified in D10.7.6,
- To overcome resistance of soil that cannot be counted upon to provide axial or lateral resistance throughout the design life of the structure, e.g., material subject to scour, or material subject to downdrag, and
- To obtain the required nominal bearing resistance.

Maximum allowable stress values recommended by various engineers, manufacturers and contractors may differ considerably, sometimes by a factor of 2. The values of maximum permissible driving stress presented reflect, where possible, past experience in the Commonwealth regarding driving stresses, and, in general, tend to fall in the median area within the range of recommended values for

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the project construction phase. When conducted during the construction phase, the drivability analysis shall be conducted using the contractor's proposed driving system. This information should be supplied by the contractor. This drivability analysis should be used to determine if the contractor's proposed driving system is capable of driving the pile to the maximum resistance anticipated without exceeding the factored structural resistance available, i.e., σ_{dr} .

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each pile type.

The designer should be aware that under certain conditions pile driveability tends to be controlled by phenomena not accurately predicted by wave equation analysis of driving stress. Piles driven through extremely loose or soft soil to point bearing on competent rock may experience localized buckling due to lack of lateral support and strong wave reflection from the pile tip. Also, piles driven into or through buried debris or boulders may experience damage due to high localized stresses where obstructions are first encountered.

10.7.8.2P Evaluation of Driveability

Preliminary evaluation of driveability shall be made by wave equation analysis using the procedure specified in D10.7.3.8.4. Driving stresses, transmitted hammer energy and developed pile capacity shall be verified during test pile construction. Changes in driving equipment, procedures, or driving criteria may be directed by the Department if monitored driving parameters vary appreciably from those assumed in the preliminary analyses.

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C10.7.8.2P

Wave equation analysis provides the most accurate evaluation of driving stresses, hammer energy and static pile strength available to the designer prior to the actual field testing and/or dynamic monitoring. Although appreciable predictive inaccuracies may occur (depending upon the particular program used, accuracy of assumed hammer efficiencies, damping factors) in wave equation analyses, they generally provide suitable information for near-optimization of pile driveability. The most common cause of inaccuracy in predicting pile driveability by wave equation analysis is use of inaccurate hammer efficiencies. Currently available wave equation programs incorporate optimistic values of hammer efficiency (typical values are 95% for diesel and 80% for air/steam hammers), which in turn lead to higher predicted transmitted hammer energies and driving stresses than are observed in the field. Use of more realistic hammer efficiencies in wave equation analyses will provide more accurate evaluation of pile driveability, and is, thus, recommended. A FHWA report, Vanikar (1985), presents average measured efficiencies of 72% for diesel, 67% for single-acting air/steam and 50% for double-acting air/steam hammers.

A driveability analysis begins with the selection of a pile type (or types) which is suitable for the soil and rock conditions present at the job site(s), and which has a structural capacity (at the time of driving) approximately 10 to 20% greater than the design capacity. A hammer is then selected which should develop optimum driving stresses in the pile. As a starting point, the following guidelines may be used to select a rated hammer energy for preliminary analysis:

- (a) Steel H-Piles: 12,000 ft-lbs
- (b) Concrete Piles (solid section): 12,000 ft-lbs
- (c) Timber Piles: 7000 ft-lbs

After choosing a pile/hammer combination, an initial wave equation analysis should be performed in accordance with D10.7.3.8.4 to evaluate the feasibility of the

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10.7.8.3P Driving Criteria

Driving shall be in accordance with Publication 408, Section 1005. Test piles shall be driven to absolute refusal. If a test pile does not achieve absolute refusal within 2 ft. of the estimated tip elevation, driving should be stopped and the Chief Bridge Engineer shall be contacted. If absolute refusal is not obtained, special studies may be required during construction. Past experience, actual driving data, wave equation analyses and possible load tests shall be correlated to obtain driving parameters.

10.7.8.4P Geologic Conditions Affecting Driveability

The presence of boulders or construction debris, buried concrete slabs, or hard layers (above a required bearing strata) may either damage piles or hinder advancement to a required elevation or bearing stratum. Such geologic conditions shall be disclosed by the subsurface investigation. Measures to increase pile driveability shall be taken in accordance with D10.7.8.5P appropriate to the location, size, or thickness of the obstructing feature(s).

10.7.8.5P Methods Of Increasing Driveability

Increasing driveability is generally achieved by either increasing the strength or stiffness of the pile (or portions thereof) or by penetrating an obstruction prior to pile installation. Steel tip protection shall be used to increase the strength of pile tips where moderate or severe damage potential conditions exist. Tip protection shall be required for all point bearing and end bearing piles driven into bedrock. Where piles must penetrate a dense layer of appreciable thickness to obtain a specified tip elevation or bearing stratum, the layer can be penetrated by jetting during pile installation or by predrilling a pilot hole prior to pile installation.

10.7.9 Probe Piles

The following shall supplement A10.7.9.

When piles are specified, each substructure unit must

preliminary pile/hammer combination. The results of the initial wave equation analysis will probably indicate that pile driveability is not optimized. Successive analyses are then conducted altering the relevant parameters as required to meet the criteria for optimized driveability.

C10.7.8.4P

In most cases, geologic conditions which will influence pile driveability are readily discernible from boring records; however, in some instances subsurface obstructions are more difficult to recognize. For example, the presence of boulders immediately overlying bedrock may be misinterpreted as a layer of fractured rock overlying sound rock. The presence of buried foundation slabs may be missed, or misinterpreted as cobbles, boulders, etc., if the exploratory drilling personnel and recovered samples are not closely monitored.

C10.7.8.5P

Increasing the stiffness of the pile section is the most common method of increasing driveability when wave equation analysis indicates overstressing will occur with a particular hammer/pile combination. When potentially damage-causing obstructions such as boulders, foundation slabs, or construction debris must be penetrated, steel tip protection will generally provide adequate protection. However, predrilling may be necessary in cases in which obstructions are unusually massive or located relatively near the ground surface where piling will have little lateral support when they are encountered. Because extensive predrilling through rock/obstructions adds considerably to foundation expense, consideration should be given to eliminating predrilling in conjunction with lower pile strengths to allow for some damage or using drilled shaft foundations.

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have at least one test pile and no less than one test pile for each 50 ft. of footing length. Additional test piles may be specified if test borings indicate that irregular pile lengths could be anticipated. In special cases in which the soil conditions are relatively uniform, one test pile for each 100 ft. of footing length will be acceptable. The specified length of test piles shall be the same as the estimated lengths of bearing piles when driven to bedrock or dense end bearing strata. For friction piles, test pile lengths shall be based on driving to absolute refusal. Test piles may be used as bearing piles, if approved.

The lengths of test piles shall be based on geotechnical data and analysis and/or previous experience (e.g., actual driving and/or load test data), if available. See D10.7.8.3P for driving criteria.

10.7.10P Evaluation of Predetermined Tip Elevations**10.7.10.1P General**

Predetermined tip elevations shall be determined in accordance with PP1.7.5 and Publication 408, Section 1005.

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C10.7.10.1P

Predetermined tip elevations are determined after driving test and/or load test piles at the foundation location. These elevations, thus determined, reflect an estimate of the length of pile embedment required to develop the nominal axial pile resistance used in design. The pile embedment length is based on the load carrying mechanism (point-bearing, end-bearing, or friction) used to develop the resistance of the pile. Because of variations in soil and rock conditions existing across a project site, tip elevations of installed piles will vary from the predetermined tip elevation. It is, therefore, necessary to interpret the data obtained from the driving and/or load testing of piles, and to evaluate the acceptability of bearing piles attaining the specified driving criterion above the predetermined tip elevation.

Guidelines are presented below for evaluating predetermined tip elevations from test and load test piles and for evaluating bearing piles which do not attain the predetermined tip elevation. Because each piling project and the subsurface conditions encountered are unique, the following guidelines should be applied with an understanding of the special conditions relating to the particular projects.

- (a) Point-Bearing Piles - Predetermined tip elevations for point bearing piles should be readily apparent by the behavior of the hammer and the observed driving resistance. The hammer will quickly approach optimum performance levels as the bearing layer is reached and may experience cylinder lift or other erratic behavior if refusal is achieved very quickly at full throttle.

If boulders are known or suspected to be present at a piling location, care shall be taken to ensure that

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a valid predetermined tip elevation is established. If the boulders are located close to the bearing stratum, it is often difficult to determine whether a pile has reached the bearing stratum or has encountered a boulder. The number of test and/or load test piles should be increased in areas where boulders overlie the bearing stratum. The engineer shall evaluate the driving and/or testing results to determine if bearing piles may be expected to penetrate the boulders and reach the bearing stratum. If a significant number of piles fail to penetrate the boulders, the engineer shall evaluate several factors before proceeding with piling installation. This may require reduction of the design resistance or revision of the driving criteria.

- (b) End Bearing Piles - End bearing piles typically reach refusal more gradually than point bearing piles. End bearing piles are typically driven to rock through a relatively thin layer of dense soils or driven a relatively short distance into a layer of very dense soil or soft rock. In the case of end bearing piles driven to competent rock, the driving resistance will typically increase to refusal quickly upon reaching bedrock. In this case, the predetermined tip elevation usually reflects the estimated top of rock. End bearing piles driven into very dense soil layers or soft rock will exhibit driving resistances which increase gradually to refusal. In these cases, the predetermined tip elevation reflects the minimum penetration into the bearing stratum needed to develop the required resistance. If load testing is performed, the piles penetrating the least into the bearing stratum shall be tested. This will allow strength verification for piles representative of the least bearing stratum embedments to be anticipated during driving of bearing piles. If boulders are encountered above or within the bearing stratum, refer to the guidelines above for point bearing piles.

- (c) Friction Piles - The driving resistance for friction piles increases gradually as the pile penetrates into the bearing stratum. Typically, the predetermined tip elevation reflects the elevation of the top of the bearing stratum minus the length of bearing stratum embedment at which test or load test piles achieve the required resistance.

Monitoring hammer performance during installation of test and/or load test piles provides a basis of comparison useful in evaluating piles which attain the specified driving criterion above the predetermined tip elevation. Thus, it is important that as much information as possible regarding hammer performance be obtained during

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10.7.10.2P Evaluation of Bearing Piles Not Reaching the Predetermined Tip Elevation

The resistance of piles not reaching the predetermined tip elevation shall be reevaluated based on available subsurface information and driving records.

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the installation of test and load test piles.

C10.7.10.2P

Very few project sites exhibit such uniform soil and rock conditions, and few pile hammers operate at such uniform levels that all bearing piles driven at the site to a particular criterion will attain the predetermined tip elevation.

The following procedures can be used to evaluate the suitability of piles not reaching the predetermined tip elevation.

a. Point Bearing Piles

1. Review Subsurface Information - Review subsurface information to determine the variation in the top of rock elevation. The pile may be acceptable if the tip elevation is within the variation in rock elevations noted during the investigation. If boulders are present, the pile may be unacceptable, pending the results of additional considerations below.
2. Review Driving Record of Suspect Pile - The driving resistance of the suspect pile should be compared to the driving resistance of adjacent piles. If the driving resistance increased rapidly over the last few feet of driving for the suspect pile, as well as adjacent piles, the driving record of the suspect pile would indicate cause for special concern. If the suspect pile exhibited a gradually increasing driving resistance in contrast to adjacent piles, it is likely that the suspect pile did not reach the same bearing stratum.
3. Review Driving Records of Adjacent Piles - Check the driving records of nearby piles with regard to the tip elevations achieved. If a trend toward higher tip elevations has occurred in the direction of the suspect pile, the top of bearing stratum elevation may be increasing in elevation, such that the suspect pile may have reached the intended stratum.
4. Drive Additional Piles - It may be prudent to reserve judgment on the suspect pile until several piles have been driven adjacent to it in locations which place the suspect between piles previously driven and the additional piles. If the additional piles behave similarly to most previously driven piles, the suspect pile is likely not acceptable. If the additional piles compare closely to the suspect pile, the suspect pile may be acceptable.

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b. End Bearing Piles

1. Review Subsurface Information - If appreciable variations in elevation of the bearing stratum are indicated, the suspect pile tip elevation may be reflecting such a variation. Where end bearing piles obtain a relatively large proportion of their axial resistance from skin friction in the lower portion of the pile, tip elevations may be sensitive to variations in the density or stiffness of the bearing stratum which may result in axial resistance development at lesser embedments (i.e., higher tip elevations).
2. Review Driving Record of Suspect Pile - Compare the driving resistance of the suspect pile with those of other piles. An end bearing pile usually exhibits relatively low driving resistances until the bearing stratum is reached. As the pile advances into the bearing stratum, driving resistance increases rapidly, which is an indicator of the top of bearing stratum elevation. If the driving resistance of the suspect pile increased at a higher elevation than adjacent piles, the suspect pile may have an embedment length in the bearing stratum similar to adjacent piles which attained the predetermined tip elevation.
3. Review Driving Records of Adjacent Piles - Check for a trend towards higher tip elevations.
4. Drive Additional Piles - Observe the behavior of several additional piles driven.
5. Check Hammer Performance - Review the records of hammer performance kept during driving of the suspect pile. If hammer performance (e.g., bounce chamber pressure, blow rate, stroke, etc., depending on hammer type and degree of instrumentation) was appreciably lower during driving of the suspect pile than during driving of adjacent piles, then the higher tip elevation may be a result of low energy input. The pile shall be redriven at optimum hammer performance to achieve the predetermined tip elevation.
6. Check Suspect Pile Axial Resistance by Dynamic Monitoring - If the suspect pile was monitored during initial driving, the estimated axial resistance may be compared to the estimated axial resistance of piles reaching the predetermined tip elevation. If the pile was not monitored when first driven, it may be monitored while being redriven. When monitoring piles being redriven, make certain that hammer performance has reached optimum levels before stopping redriving, since less than optimum

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hammer performance will likely be reflected by reduced nominal strength estimates (see D10.7.3.8.3dP, Redriving). A favorable comparison between the nominal strength of the suspect pile and the strength of piles driven to the predetermined tip elevation indicates the acceptability of the suspect pile.

7. Check Potential for Temporary Increase in Driving Resistance - When piles are driven into dense silts or glacial tills below the water table, or into soft shales, siltstones or claystones, the driving resistance may be temporarily increased due to the behavior of the bearing stratum. After a 24-hour period has elapsed following initial driving, driving resistances have been observed to decrease by half as piles are redriven. It is possible that a pile failing to achieve the predetermined tip elevation in soil or rock conditions, as described above, encountered temporary resistance, rather than actual bearing. Suspect piles may achieve the predetermined tip elevation if redriven a day or more following initial driving.

c. Friction Piles

1. Check Hammer Performance - Compare the level of performance to that observed during test pile driving and driving of nearby bearing piles. If the suspect pile was installed at lower hammer energies, it should be redriven at optimum energy to attain the predetermined tip elevation. Embedment lengths of friction piles are often very sensitive to fluctuations in hammer performance.
2. Review Subsurface Information - Where the bearing stratum is encountered at a higher elevation than at previous piling locations, or where a greater density was indicated during subsurface investigation, friction piles may be expected to attain a specified driving criterion at corresponding higher elevations.
3. Review Driving Record of Suspect Pile - If the suspect pile encountered appreciable driving resistance at a higher elevation than piles which attained the predetermined tip elevation, the suspect pile may have an embedment length in suitable material similar to piles driven to the predetermined tip elevation.
4. Drive Additional Piles - Observe the driving behavior and tip elevations of additional piles to evaluate the possible continuation of conditions encountered by suspect pile.

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5. Review Driving Records of Adjacent Piles - Check for a trend toward higher tip elevations.
6. Check Suspect Pile Axial Resistance by Dynamic Monitoring - If dynamic monitoring equipment is being used on the project, check the axial resistance of the suspect pile.
7. Check Potential for Temporary Increase in Driving Resistance - If soil conditions "artificially" elevate, driving resistances at the end of initial driving, redriving the suspect pile after a waiting period may advance the pile to the predetermined tip elevation.

10.7.11P Foundation Submission

As part of the foundation approval process, a foundation submission letter shall be submitted to the Department for use in foundation review, in compliance with PP1.9.4 and D11.4.4P. The letter shall include the basis for the pile capacity (e.g., load tests, wave equation analyses, dynamic monitoring, past experience) and the relevance of field conditions and construction procedures used to develop the pile capacity. In addition to the basic requirements of PP1.9.4 and D11.4.4P, the foundation submission letter shall include the following, as a minimum:

- a. Pile type and size (including alternates),
- b. Geotechnical axial pile capacity,
- c. Structural pile capacity,
- d. Basis for pile capacity determination,
- e. Bottom of pile cap elevation,
- f. Estimated tip elevations and maximum pile lengths,
- g. Description of anticipated bearing stratum,
- h. Anticipated load carrying mechanism (point bearing, end bearing, or friction),
- i. Relevant soil and/or rock conditions,
- j. If requested, in special and/or unusual situations, preliminary wave equation analysis, for a trial driving system including soils and driving system input parameters,
- k. Where applicable, foundation type of existing structure and other pertinent information,

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- l. Downdrag analysis, if anticipated,
- m. COM624P or LPILE 5.0 analysis results and lateral pile capacities, and
- n. Evaluation of corrosion potential and design measures for protection against deterioration.
- o. Indicate if soils are classified as Site Class E or F.

10.8 DRILLED SHAFTS**10.8.1 General****10.8.1.1 Scope**

The following shall supplement A10.8.1.1.

Drilled shafts shall be considered for foundation support when spread footings cannot be founded on suitable soil or rock strata within a reasonable depth (e.g., 10 ft.), in areas of karst conditions, and when piles are not economically feasible due to high loads or obstructions to driving. Drilled shafts shall also be considered when high lateral or uplift loads must be resisted and deformation tolerances are small, or as a direct support element for columns used as pier bents. As an alternate to drilled shafts, foundation pedestals or columns may be considered when suitable rock bearing is within 10 ft. of the ground surface (see D10.6.5.2P).

Drilled shafts not founded on or socketed into rock are generally not permitted by the Department and, if used, must be approved by the Chief Bridge Engineer.

The terminology used in the design of drilled shafts is shown in Figure 10.8.1.1-1P.

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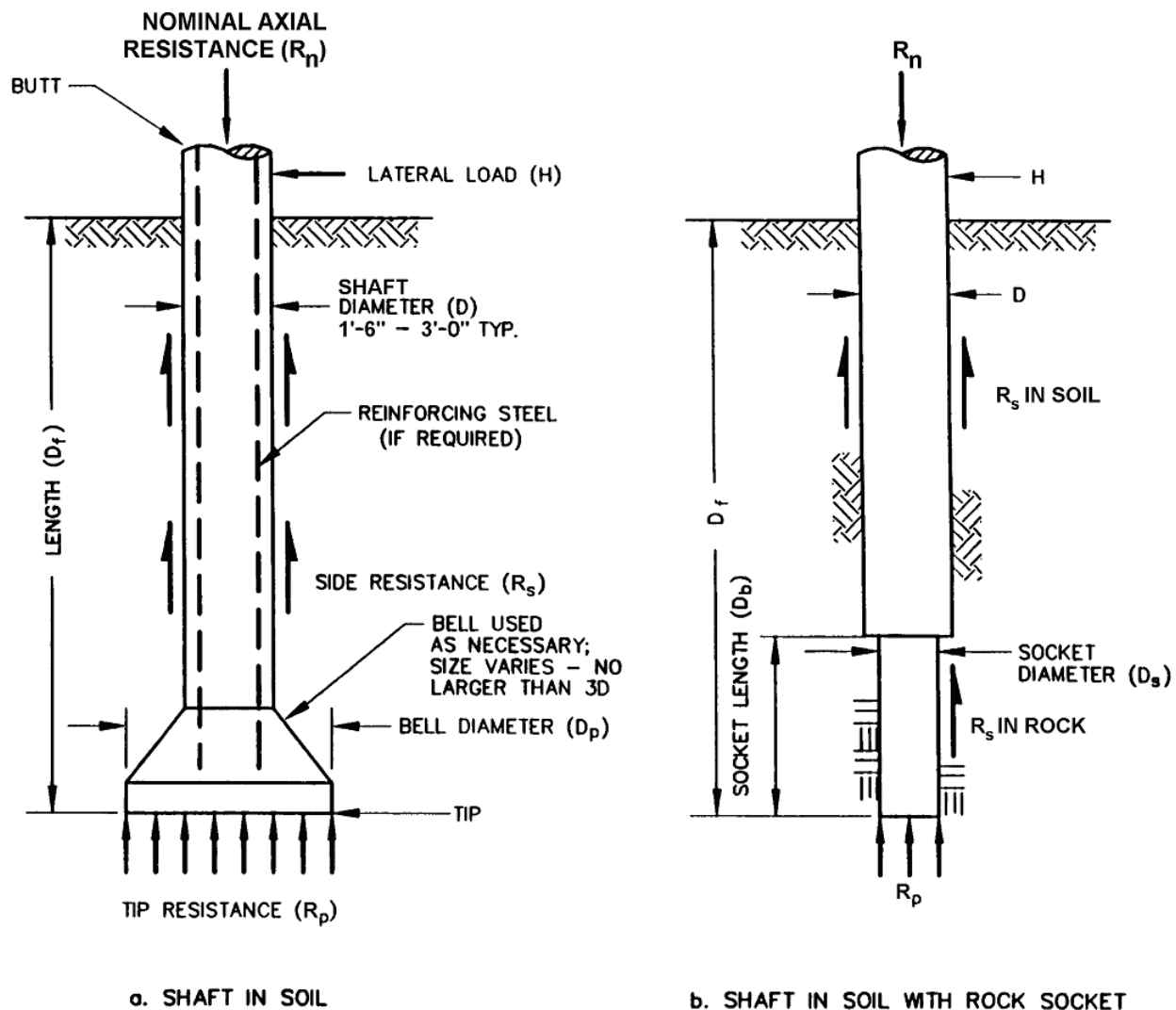


Figure 10.8.1.1-1P - Design Terminology for Drilled Shaft Foundations

10.8.1.2 Shaft Spacing, Clearance, and Embedment into Cap

The following shall supplement A10.8.1.2.

Shaft reinforcement shall extend sufficiently into the cap to overlap horizontal cap reinforcement in order to develop the required stress transfer.

10.8.1.3 Shaft Diameter and Enlarged Bases

The following shall supplement A10.8.1.3.

For drilled shaft requiring inspection, the shaft dimensions shall not be less than 3-feet. All shafts shall be sized in 6-inch increments with a minimum shaft diameter of 1'-6". Inclined or battered shafts shall not be used without the prior approval of the Chief Bridge Engineer. The diameter of shafts with rock sockets shall be sized a minimum of 6-inch larger than the diameter of the socket.

C10.8.1.3

The following shall supplement AC10.8.1.3.

Most drilling tools and casings used in the United States are sized in 6-inch size intervals. Therefore, unless special project requirements dictate the use of unconventional shaft dimensions, it is most economical to size shaft diameters in 6-inch increments.

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The diameter of columns supported by shafts shall be less than D . See Figure 10.8.1.3-1P for dimensional guidelines.

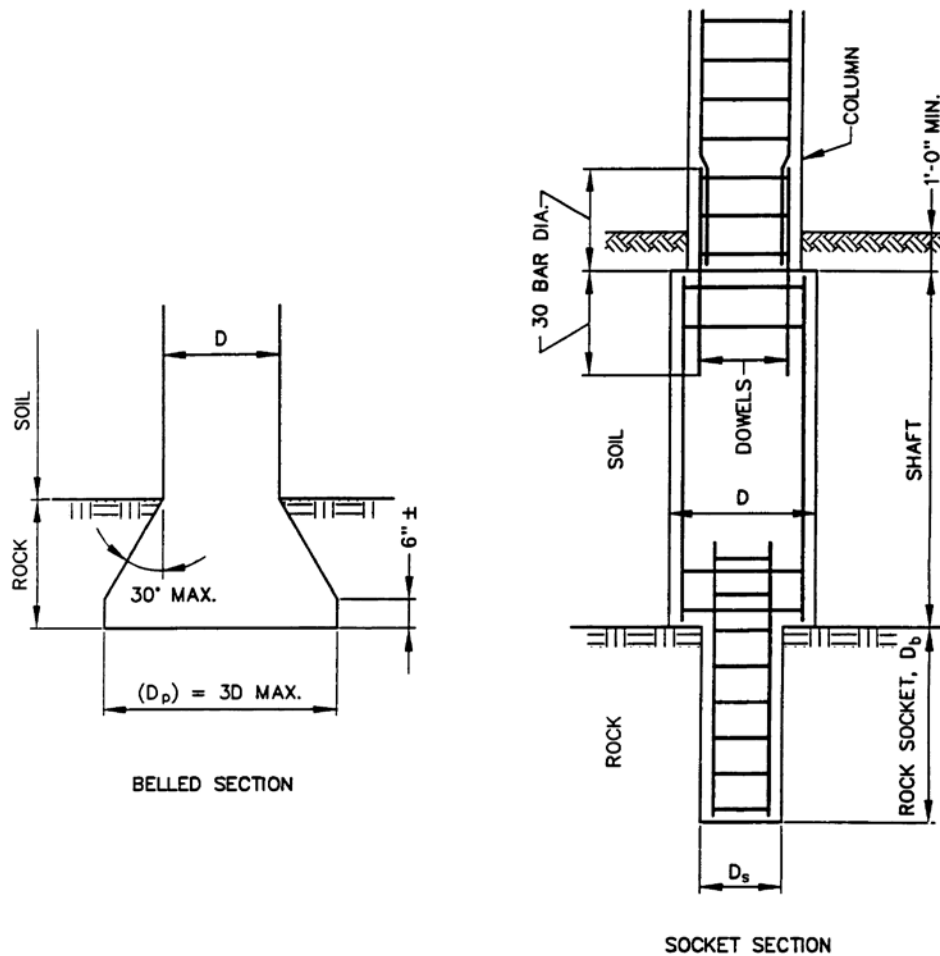


Figure 10.8.1.3-1P - Dimensional Guidelines for Drilled Shafts

10.8.1.5 Drilled Shaft Resistance

The following shall supplement A10.8.1.5.

Drilled shafts shall be constructed using the dry, casing or wet method of construction or a combination of these methods.

In every case, hole excavation, concrete placement and all other aspects of shaft construction shall be performed in conformance with the provisions of the specifications, Publication 408 and applicable Special Provisions.

10.8.1.6 Determination of Shaft Loads

10.8.1.6.3 Uplift

The following shall supplement A10.8.1.6.3.

Tension in drilled shafts is not permitted at Service Limit States. At Strength Limit States, the uplift capacity

C10.8.1.5

The following shall supplement AC10.8.1.5.

Past practice in the Commonwealth is that temporary or permanent casing is used to support the sides of the drilled shaft hole.

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may be taken as 10 percent of the axial structural capacity.

10.8.3 Strength Limit State Design**10.8.3.3 Scour**

The following shall supplement A10.8.3.3.

Scour investigations and design of bridge structure foundations to resist scour shall be in accordance with PP7.2.

10.8.3.5 Nominal Axial Compression Resistance of Single Drilled Shafts**C10.8.3.5**

The following shall supplement AC10.8.3.5.

The load transfer and deformation characteristics of drilled shafts are a function of the ratios of shaft depth to diameter (D_f/D), the shaft diameter (D) to base diameter (D_p), and the relative stiffness between the shaft and the soil. The following observations are applicable:

1. As D_f/D and soil stiffness decrease, the proportion of load supported by the tip (R_p) increases.
2. Shafts with large values of D_f/D develop appreciable R_s before mobilizing any R_p .
3. Butt displacements of the order of 8 to 10% of D are necessary to mobilize R_p in cohesionless soils and stiff clays.
4. The ultimate shaft resistance (R_s) is typically fully mobilized at displacements of about 0.2 to 0.4 in.

Drained analyses using effective soil shear strengths (c' and ϕ_f) are appropriate for cohesionless soils and most rock types, and for cohesive soils which are permitted to drain and consolidate under loading. Such analyses are appropriate for bridge pier foundations which are subjected to large dead loads and relatively small live loads. Undrained loading should be considered where very high live loads are anticipated in cohesive soils. Live loads are of short duration, and do not provide sufficient time for draining of cohesive soils.

10.8.3.5.1 Estimation of Drilled Shaft Resistance in Cohesive Soils**10.8.3.5.1c Tip Resistance****C10.8.3.5.1c**

The following shall supplement AC10.8.3.5.1c.

Use of enlarged bases requires the prior approval of the Chief Bridge Engineer. The tip resistance of an enlarged base shall be determined assuming that the entire base area is effective in transferring load. Allowance of full effectiveness of the enlarged base shall be permitted only

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*10.8.3.5.2 Estimation of Drilled Shaft Resistance in Cohesionless Soils**10.8.3.5.2b Side Resistance*

The following shall supplement A10.8.3.5.2b.

A correlation between N_{60} blow count, friction angle and unit weight of material is provided in Table 10.8.3.5.2b-1P.

Table 10.8.3.5.2b-1P - Friction Angles and Unit Weights of Sands

CONSISTENCY	ϕ_f	N_{60}	$\gamma(\text{kcf})$
Very Loose	25°- 30°	0 - 4	0.070-0.100
Loose	27°- 32°	4-10	0.090-0.115
Medium	30°- 35°	10-30	0.110-0.130
Dense	35°- 40°	30-50	0.110-0.140
Very Dense	38°- 43°	>50	0.130-0.150

10.8.3.5.3 Shafts in Strong Soil Overlying Weaker Compressible Soil

The following shall supplement A10.8.3.5.3.

Where the tip of a shaft could bear on a thin firm soil layer underlain by a softer soil unit, the shaft shall be extended through the softer soil unit to eliminate the potential for a punching shear failure into the softer soil deposit.

*10.8.3.5.4 Estimation of Drilled Shaft Resistance in Rock**10.8.3.5.4a General*

The following shall replace the 3rd bullet in A10.8.3.5.4a.

- A combination of both, with approval of the Chief Bridge Engineer.

The following shall supplement A10.8.3.5.4a.

The side resistance from overlying soil deposits and

when cleaning of the bottom of the drilled excavation is specified and can be acceptably completed before concrete placement.

An enlarged base may be used at the tip of a shaft to increase the tip bearing area, or to provide additional resistance to uplift loads. Due to the difficulty of excavation and support of enlarged bases, consideration should be given instead to extending the shaft to a greater depth to provide additional resistance. This avoids the construction difficulties and high additional cost of shafts with enlarged bases relative to straight-sided shafts.

C10.8.3.5.3

The following shall supplement AC10.8.3.5.3.

Punching shear failure is a failure mode typically associated with drilled shafts bearing on soils which behave plastically, but it is also of concern where shafts bear on a thin firm soil layer underlain by a softer deposit. In such cases, the influence of the bearing load at the surface of the soft layer shall be analyzed.

C10.8.3.5.4a

The following shall supplement AC10.8.3.5.4a.

Rock stratification should be considered in the design of rock sockets as follows:

- Sockets embedded in alternating layers of weak and strong rock should be designed using the strength of the weaker rock.

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weak rock shall be ignored.

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- The side resistance provided by soft or weathered rock should be neglected in determining the required socket length where a socket extends into more competent underlying rock. Rock is defined as soft when the uniaxial compressive strength of the weaker rock is less than 20% of that of the stronger rock or weathered when the RQD is less than 20%.
- Where the tip of a shaft would bear on thin rigid rock strata underlain by a weaker unit, the shaft should be extended into or through the weaker unit (depending on load capacity or deformation requirements) to eliminate the potential for failure due to flexural tension or punching failure of the thin rigid stratum.
- Shafts designed to bear on strata in which the rock surface is inclined should extend to a sufficient depth to ensure that the shaft tip is fully bearing on the rock.
- Shafts designed to bear on rock strata in which bedding planes are not perpendicular to the shaft axis should extend a minimum depth of 2B into the dipping strata to minimize the potential for shear failure along natural bedding planes and other slippage surfaces associated with stratification

10.8.3.5.4b Side Resistance

The following shall supplement A10.8.3.5.4b.

A correlation between RQD and E_m/E_i based on joints is provided in Table 10.8.3.5.4b-2P.

Table 10.8.3.5.4b-2P Estimation of E_m based on RQD
(after O'Neill and Reese, 1999)

RQD (percent)	E_m/E_i	
	Closed Joints	Open Joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

10.8.3.5.4c Tip Resistance

C10.8.3.5.4c

The following shall replace the 4th paragraph of AC10.8.3.5.4c.

Eq. A10.8.3.5.4c-2 is a lower bound solution for bearing resistance for a drilled shaft bearing on or socketed in a fractured rock mass. This method is appropriate for rock with joints that are not necessarily oriented preferentially and the joints may be open, closed, or filled with weathered material. Load testing will likely indicate

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10.8.3.5.6 *Shaft Load Test*

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higher tip resistance than that calculated using Eq. A10.8.3.5.4c-2. Resistance factors for this method have not been developed and must therefore be estimated by the designer with review and approval by the Chief Bridge Engineer.

C10.8.3.5.6

The following shall supplement C10.8.3.5.6.

For large diameter shafts, where conventional reaction frames become unmanageably large, load testing using Osterberg load cells (O-Cells) may be considered. An advantage of an O-Cell load test is that the load is applied from the bottom of the rock socket and data on the load carrying capacity of the rock socket can be directly obtained. An O-Cell load test was performed on a large diameter technique shaft at a site located at the borders of Fayette and Somerset Counties. The rock strata providing bearing for the test shaft is the Freeport Formation. A summary of the drilled shaft design and test results is presented in Table C10.8.3.5.6-1P.

The maximum test load on the rock socket in side shear was 11,486 kips, over 2.6 times the anticipated maximum factored axial load. The estimated average ultimate net unit side shear over the full 20-foot long rock socket was on the order of 20.6 ksf. This is in good agreement with the design average ultimate unit side resistance of 19.2 ksf. The O-Cell results indicated that the ultimate net unit side shear of the stronger siltstone at the top and bottom of the rock socket was higher than the calculated design value. However, the O-Cell results indicated that, as the test load increased, no load was carried in the weaker argillaceous siltstone sandwiched between the stronger siltstone strata.

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Table C10.8.3.5.6-1P - Summary of Drilled Shaft Design and Osterberg Cell Load Test Results
Fayette and Somerset Counties

SHAFT DATA			
Shaft Diameter = 9 ft. Rock Socket Diameter = 8.5 ft. Rock Socket Length = 20 ft. Maximum Factored Axial Load = 4347 kips - Design socket length controlled by lateral capacity - Design capacity is based on side resistance only; no end bearing			
Rock Stratum	Upper Siltstone	Argillaceous Siltstone	Lower Siltstone
Approximate Thickness in Socket of Test Shaft, ft.	8.0	4.0	8.0
Average RQD, %	71	89	90
Average RMR	56	65	64
Design Compressive Strength, psi	3500 ⁽¹⁾	1018	3500 ⁽¹⁾
Design Ultimate Unit Side Resistance, ksf	12.2	11.4	12.2
Design Factored Unit Side Resistance ⁽²⁾ , ksf	11.6	6.2	11.6
Estimated Ultimate Unit Side Shear from Load Test, ksf	27.0	0	24.5
(1) Rock compressive strength is greater than concrete compressive strength of 3500 psi. 3500 psi used in design. (2) Resistance Factor = 0.55.			

10.8.3.7 Uplift Resistance

The following shall supplement A10.8.3.7.

Uplift resistance shall not be used without approval of the Chief Bridge Engineer.

*10.8.3.7.2 Uplift Resistance of Single Drilled Shaft**C10.8.3.7.2*

The following shall supplement AC10.8.3.7.2.

The effect of an enlarged base on uplift displacements is usually ignored because mobilization of any uplift resistance associated with enlarged bases typically occurs at displacements outside the tolerable limits of shaft displacement.

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10.8.3.8 Nominal Horizontal Resistance of Shaft and Shaft Groups

The following shall supplement A10.8.3.8.

The design of laterally loaded drilled shafts shall account for the effects of soil layering, variable groundwater level, loss of lateral ground support (e.g., scour), cyclic loading, combined axial and lateral loading and sloping ground.

The final design of laterally loaded drilled shafts shall be based on the results of COM624P computer analyses, Wang and Reese (1993), and Reese (1984) or LPILE 5.0 (see ENSOFT, Inc. 2004 for LPILE 5.0). Other methods of analysis to evaluate the nominal horizontal resistance or deflection of laterally loaded shafts may be used for preliminary design only as a means to determine approximate shaft dimensions.

The effects of group action shall be considered in the design of laterally loaded drilled shafts. Refer to D10.7.3.12.3P for additional information on group lateral load resistance

10.8.3.9 Shaft Structural Resistance*10.8.3.9.3 Reinforcement*

The following shall supplement A10.8.3.9.3.

The minimum clear distance between reinforcement shall not be less than 1.5 times the bar diameter nor 1.5 times the maximum aggregate size, provided the concrete can be vibrated. If the concrete cannot be vibrated, the minimum distance between reinforcement shall not be less than three times the bar diameter nor three times the maximum aggregate size. See Figure D10.8.1.3-1P for guidelines for layout of steel reinforcement including minimum bar lap.

Permanent steel casing, if used, shall have a thickness sufficient to withstand installation stresses, stresses due to lateral earth pressure and groundwater, and corrosion.

10.8.5P Foundation Submission

As part of the foundation approval process, a foundation submission letter shall be submitted to the Department for use in foundation review, in compliance with PP1.9.4 and D11.4.4P. The letter shall include the basis for the drilled shaft resistance (e.g., load tests, resistance analyses and past experience) and the relevance of field conditions and construction procedures used to develop the shaft resistance. In addition to the basic requirements of PP1.9.4 and D11.4.4P, the foundation submission letter shall include the following, as a minimum:

- a. Shaft type and size (including alternates)

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C10.8.3.8

The following shall supplement AC10.8.3.8.

Refer to O'Neill and Reese (1999) for methods of analysis to estimate nominal resistance of laterally loaded drilled shafts for preliminary design.

The major portion of lateral load resistance is mobilized within a depth equal to the five to eight shaft diameters from the ground surface.

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- b. Geotechnical axial shaft resistance
- c. Structural shaft resistance
- d. Basis for shaft resistance determination
- e. Bottom of cap elevation
- f. Estimated tip elevations and maximum shaft lengths
- g. Description of anticipated bearing stratum
- h. Anticipated load carrying mechanism (point bearing, end bearing, or friction)
- i. Relevant soil and/or rock conditions
- j. Where applicable, foundation type of existing structure and other pertinent information
- k. Downdrag analysis, if anticipated
- l. COM624P or LPILE 5.0 analysis results and lateral shaft resistances
- m. Evaluation of corrosion potential and design measures for protection against deterioration.
- n. Indicate if soils are classified as Site Class E or F.

10.9 MICROPILES**10.9.1 General**

The following shall replace A10.9.1.

In Pennsylvania, micropiles are constructed by placing a sand-cement mortar or neat cement grout in the pile under a gravity head only.

Micropiles are to utilize steel casing that meets ASTM A-252 Grade 3, but have minimum yield strength of 80 ksi. The material may be mill secondary API drill casing. The core reinforcing bar is to have a minimum yield of 75 ksi. The neat cement grout used for the micropiles is to have a minimum 28-day unconfined compressive strength, $f'_c = 4$ ksi. Micropiles may range from 5 inches to 12 inches in diameter.

10.9.1.2 Minimum Micropile Spacing, Clearance, and Embedment into Cap

The following shall replace A10.9.1.2.

Micropile spacing, clearances and embedment into the foundation is in accordance with D10.7.1.2. Group effects must be considered for micropiles spaced closer than 30 in. center to center.

C10.9.1.2P

The factored load effect acting on any micropile in a group may be estimated using the traditional elastic strength of material procedure for a cross-section under thrust and moment.

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The center to center spacing of micropiles should be greater than 3.0 micropile diameters or the spacing required to avoid interaction between adjacent micropiles. Larger spacings may be required where drilling operations are anticipated to be difficult.

If closer spacings are required, the sequence of construction shall be specified in the contract documents, and the interaction effects between adjacent micropiles shall be evaluated.

The connection between micropiles and footings shall be designed to distribute structure loads and overturning moments to all micropiles in a group. Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by micropiles, the concrete cover at the sides of the micropiles shall be greater than 6 inches.

10.9.1.3 Micropile through Embankment Fill

The following shall supplement A10.9.1.3.

Required micropile penetration should be determined based on the resistance to vertical and lateral loads and the displacement of both the micropile and the subsurface materials.

Micropiles for trestle or pile bents shall penetrate a distance equal to at least one-third the unsupported length of the micropile.

Micropiles used to penetrate a soft or loose upper stratum overlying a hard or firm stratum, shall penetrate the firm stratum by a distance sufficient to limit movement of the micropiles per D10.5.2.2 and attain sufficient bearing resistance.

10.9.1.4 Battered Micropiles**C10.9.1.4**

The following shall supplement AC10.9.1.4.

Use of vertical micropiles to resist lateral loads shall only be considered with the approval from the Chief Bridge Engineer.

10.9.1.5 Micropile Design Requirements**C10.9.1.5**

The following shall supplement A10.9.1.5.

The resistance of micropiles should be determined by static analysis methods based on soil structure interaction. The resistance of micropiles should be determined through a subsurface investigation, laboratory and/or in-situ tests, analytical methods, micropile load tests, and reference to the past performance of micropiles in similar ground conditions. Consideration shall also be given to:

- The difference between the resistance of a single micropile and that of a group of micropiles;

The following shall supplement AC10.9.1.5.

Micropiles which develop capacity in soil shall only be considered with approval from the Chief Bridge Engineer.

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- The capacity of the underlying strata to support the load of the micropile group;
- The effects of groundwater level consistent with that used to calculate load effects. The effect of hydrostatic pressure shall be considered;
- The possibility of scour and its effect; and
- The transmission of forces, such as negative skin friction or downdrag forces, from consolidating soil.

Micropiles are to transfer their load through grout to ground friction in the bonded length without contribution of end bearing.

Micropiles shall develop capacity in rock.

Micropile foundations designed to resist uplift forces should be checked for resistance to pullout and structural ability to carry tensile stress in accordance with D10.7.1.6.3 and D10.7.3.10.

Uplift forces can be caused by lateral loads, buoyancy effects, and expansive soils. The connection of the micropile to the footing is part of its structural ability to resist uplift and should also be investigated.

10.9.1.7P Estimated Micropile Lengths

Estimated micropile lengths for each substructure shall be shown on the plans in accordance with PP1.6.4.11, and shall be based on careful evaluation of available subsurface information, static and lateral capacity calculations, and/or past experience.

10.9.1.8P Estimated and Minimum Tip Elevations

See A10.7.6 and D10.7.6.

Estimated and minimum micropile tip elevations for each substructure should be shown on the contract plans.

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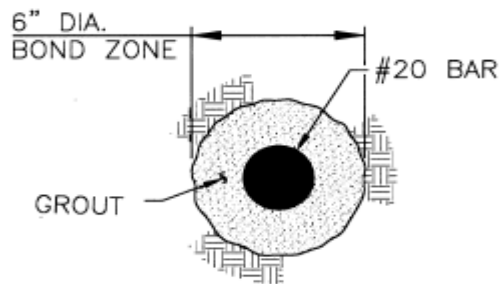
COMMENTARY

10.9.1.9P Micropiles Notes and Sketches

The following notes shall be placed on the General Notes sheet of the contract drawings:

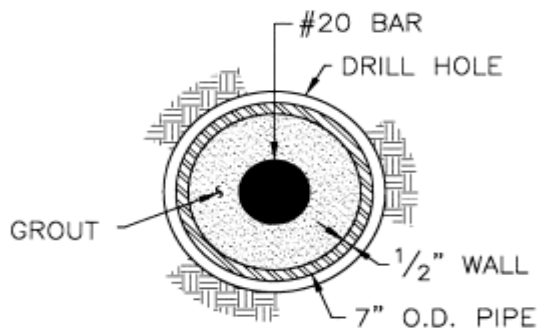
- Each individual pile bond zone must contain an accumulation of $\underline{X'}$ of rock.
- Each bond zone must be a minimum of $\underline{Y'}$ in length.
- No one soil seam in excess of 5' will be acceptable.
- No bond zone shall be terminated without 2.0' (min.) of rock at the bottom of the bond zone.
- Each bond zone must be extended as necessary to a length that includes an accumulation of $\underline{Z'}$ of rock.

{The X, Y, and Z values are based on calculations and subsurface conditions}



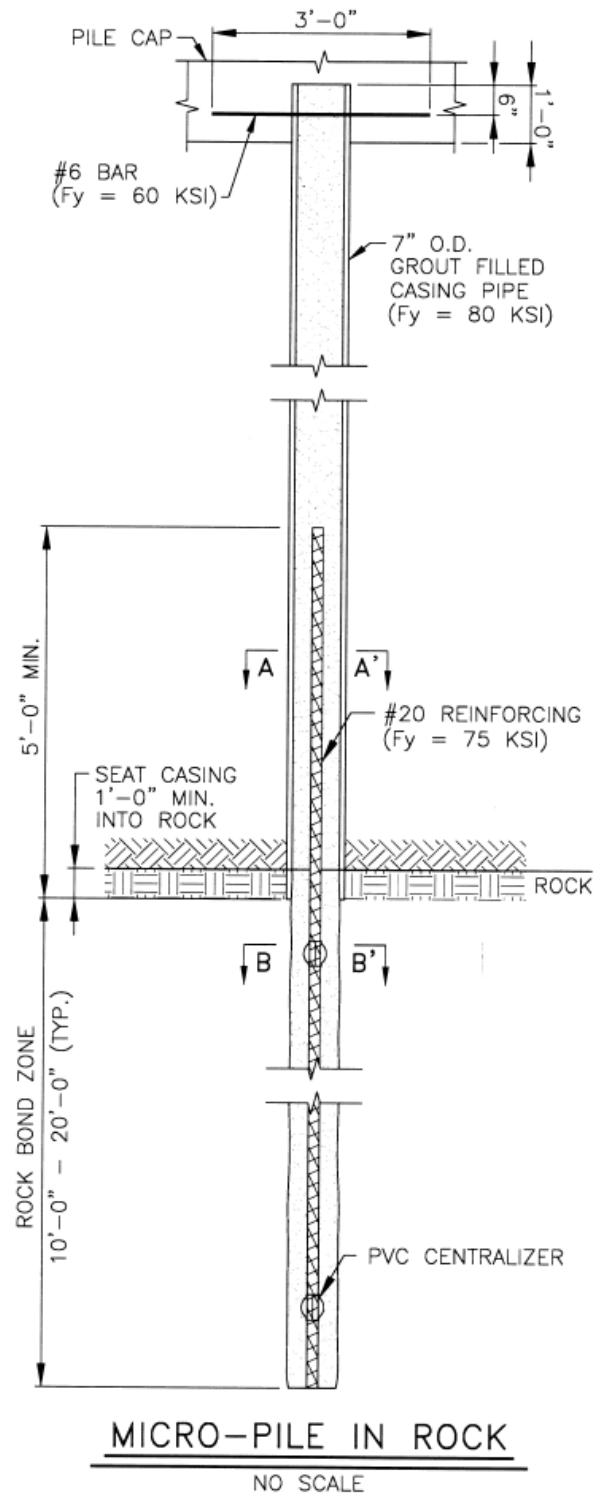
SECTION B-B'

NO SCALE



SECTION A-A'

NO SCALE



MICRO-PILE IN ROCK

NO SCALE

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10.9.2 Service Limit State Design**10.9.2.3 Settlement**

The following shall supplement A10.9.2.3.

The settlement of a micropile foundation shall not exceed the tolerable settlement, as selected according to DM-4. Refer to D10.7.2.2 for criteria for horizontal displacement.

10.9.2.4 Horizontal Micropile Foundation Movement

The following shall supplement A10.9.2.4.

See provisions of D10.7.2.4.

C10.9.2.4

The following shall supplement AC10.9.2.4.

The lateral displacement of micropile groups shall be estimated using procedures that consider soil structure interaction, in accordance with DC10.7.2.4.

10.9.2.7P Uplift

Tension in micropiles is not permitted at the service limit state.

10.9.3 Strength Limit State Design**10.9.3.1 General**

The following shall supplement A10.9.3.1.

Tip resistance shall be neglected for micropile design.

Structural resistance (compression strength) of the bonded zone generally controls the maximum load.

10.9.3.5 Nominal Axial Compression Resistance of a Single Micropile*10.9.3.5.1 General*

The following shall replace A10.9.3.5.1.

Micropiles shall be designed to resist failure of the bonded length in rock. The resistance factors for the shaft resistance (grout-to-ground bond) are specified in Table D10.5.5.2.5-1.

C10.9.3.5.1

The following shall replace AC10.9.3.5.1.

In Pennsylvania, micropiles are designed considering only grout-to-ground bond (shaft) resistance.

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10.9.3.5.2 Estimation of Grout-to-Ground Bond Resistance

C10.9.3.5.2

The following shall replace A10.9.3.5.2.

The bond resistance over the uncased, bonded length below casing tip of a micropile is computed as:

$$Q_R = \phi Q_n = \phi_s Q_s = \phi_s (\pi d_b \alpha_b D_b) \quad (10.9.3.5.2-1)$$

where:

- Q_s = nominal micropile bond resistance (kips).
- d_b = diameter of micropile drill hole through bonded length (in).
- α_b = nominal micropile grout-to-ground bond strength (ksi).
- D_b = micropile bonded length (in.).
- ϕ_s = resistance factor for bond resistance specified in Table D10.5.5.2.5-1

The bond resistance of micropiles may be based on the results of micropile load test; estimates based on a review of geologic and boring data, soil and rock samples, laboratory testing and previous experience; or estimated using published grout to ground bond guidelines. For final design, micropile capacity shall be verified through the performance of micropile load tests as described in D10.9.3.5.4.

The following shall replace AC10.9.3.5.2.

The value of nominal unit grout-to-ground bond strength, either estimated empirically or determined through load testing, is typically taken as the average value over the entire bond length.

Micropile grout-to-ground bond strength is influenced by soil and rock conditions, method of micropile drilling and installation, and grouting pressure. As a guide, Table C10.9.3.5.2-1 may be used to estimate the nominal (ultimate) unit grout-to-ground bond strength.

Table C10.9.3.5.2-1 – Summary of Typical α_b Values (Grout-to-Ground Bond) for Preliminary Micropile Design (modified after Armour, et al. 2000)

Rock Description	Typical Range of Grout-to-Ground Bond Strength (ksi)
Soft Shales (fresh-moderate fracturing, little to no weathering)	0.03 – 0.08
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	0.08 – 0.20
Limestone (fresh-moderate fracturing, little to no weathering)	0.15 – 0.30
Sandstone (fresh-moderate fracturing, little to no weathering)	0.08 – 0.25
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	0.20 – 0.61

10.9.3.5.3 Estimation of Micropile Tip Resistance in Rock

The following shall replace A10.9.3.5.3.

Tip resistance in rock shall not be considered.

10.9.3.5.4 Micropile Load Test

The following shall supplement A10.9.3.5.4.

Test micropiles shall be installed at each substructure to determine micropile installation characteristics, evaluate

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micropile capacity with depth, and establish contractor micropile order lengths. One static load test per substructure unit is to be performed. The maximum test load must be no less than two (2) times the micropile unfactored design load (DL). The number of test micropiles required may be increased in non-uniform subsurface conditions. The number of test micropiles may be reduced where previous experience exists with the same micropile type and ultimate micropile capacity, similar subsurface conditions, similar installation methods and contractor personnel experience. However, the number of test micropiles and load test may only be reduced with the Chief Bridge Engineer's approval.

10.9.3.7 Nominal Uplift Resistance of a Single Micropile

The following shall supplement A10.9.3.7.

The uplift resistance of a single micropile shall be estimated based on the bond resistance of micropiles specified in D10.9.3.5.2.

Factored uplift resistance shall be taken as:

$$Q_R = \phi Q_n = \phi_u Q_s \quad (10.9.3.7-1P)$$

where:

ϕ_u = resistance factor for uplift capacity specified in Table D10.5.5.2.5-1

Q_s = nominal micropile bond resistance (kips) (D10.9.3.5.2)

10.9.3.8 Nominal Uplift Resistance of Micropile Groups

The following shall replace A10.9.3.8.

Micropile group factored uplift resistance shall be taken as:

$$Q_R = \phi Q_n = \phi_{ug} Q_{ug} \quad (10.9.3.8-1P)$$

where:

ϕ_{ug} = Resistance factor specified in Table D10.5.5.2.5-1

Q_{ug} = Nominal uplift resistance of the group (kips)

The uplift resistance, Q_{ug} of a micropile group shall be taken as the lesser of:

- The sum of the individual micropile uplift resistances, or
- The uplift capacity of the micropile group considered as a block.

C10.9.3.7

The following shall supplement AC10.9.3.7.

The preliminary design of micropiles subjected to tension loading may be based on the estimated nominal unit grout-to-ground bond strengths presented in Table DC10.9.3.5.2-1.

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10.9.3.9 Nominal Horizontal Resistance of Micropiles and Micropile Groups

The following shall replace the first paragraph in A10.9.3.9.

Micropiles subject to lateral loads are to be designed in accordance with D10.7.3.12.2P and D6.15.1. Lateral resistance of micropile groups is to be in accordance with D10.7.3.12.3P.

10.9.3.10 Structural Resistance*10.9.3.10.2 Axial Compressive Resistance*

The following shall replace A10.9.3.10.2.

The upper, cased section of a micropile subjected to compression loading shall be designed structurally to support the full factored load on the micropile.

For micropiles extending through a weak upper soil layer, extending above ground, subject to scour, extending through mines/caves or extending through soil that may liquefy, the effect of any laterally unsupported length shall be considered in the determination of axial compression resistance.

10.9.3.10.2a Cased Length

The following shall supplement A10.9.3.10.2a.

A portion of the cased length of the micropile will consist of the casing and grout. The lower portion will contain the reinforcing bar which extends through the bonded zone.

The following shall replace the definition of f'_c in A10.9.3.10.2a.

f'_c = specified compressive strength of micropile grout at 28 days unless another age is specified. Maximum for design purposes is 4 ksi, regardless of actual grout compressive strength. (ksi)

10.9.3.10.2b Uncased Length

The following shall supplement A10.9.3.10.2b.

The uncased length consists of the bonded zone of the micropile and contains grout and reinforcing bar.

10.9.3.10.3 Axial Tension Resistance

The following shall replace A10.9.3.10.3

The upper, cased section of a micropile subjected to tension loading shall be designed structurally to support the full factored load on the micropile. The lower uncased section of a micropile subjected to tension loading shall be

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designed structurally to support the full factored load on the micropile.

10.9.3.10.3a Cased Length

C10.9.3.10.3a

The following shall supplement AC10.9.3.10.3a.

Micropiles used to resist axial tension loads in excess of 20 percent of the pipe/casing thread strength generally incorporate a full-length reinforcement bar designed to support the full factored load.

10.9.3.10.4 Plunge Length Transfer Load

C10.9.3.10.4

The following shall replace A10.9.3.10.4.

A one foot plunge length (embedment of casing into bond length) is to be provided. However, no additional load transfer due to the plunge length may be incorporated into the design.

Delete AC10.9.3.10.4.

10.9.3.10.6 Buckling and Lateral Stability

The following shall replace A10.9.3.10.6.

Micropiles that extend through water or air shall be checked for buckling in accordance with D5.13.4.7.4P

10.9.5 Corrosion and Deterioration

The following shall supplement A10.9.5.

As a minimum, the following types of deterioration shall be evaluated:

- Corrosion of steel, particularly in fill soils, low pH soils, and marine environments;
- Sulfate, chloride, and acid attack of concrete and cement grout.

Refer to D10.7.5.6P for determination of corrosive environment. Minimum corrosion protection/sacrificial thickness for micropiles shall be in accordance with D10.7.5.2P Steel Piles. For measures to protect piles against deterioration, refer to D10.7.5.

10.10 REFERENCES

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APPENDIX B10P – ESTIMATED VALUES FOR ALLOWABLE BEARING PRESSURE**B10.1P SCOPE**

Estimated values for allowable bearing pressure on rock developed for working stress design are presented in Table B10.1P-1. These values may be used for preliminary sizing of foundations.

Table B10.1P-1 Summary of Allowable Bearing Values for Rock

Lithology	Uniaxial Compressive Strength C_o (tsf)	Range of Allowable Bearing Pressure, q_{all} (tsf)				
		RQD %				
		≤ 60	70	80	90	100
Sandstone	720-1,800	5-15	15-46	30-75	40-80	55-100
Siltstone	145-575	2-5	4-15	6-25	8-35	12-50
Shale	70-370	1-4	2-10	3-15	5-25	6-30
Claystone	15-70	0.1-0.6	0.4-2	0.5-3	1-4	1-6
Limestone	1,080-2,160	8-15	25-50	45-90	65-100	100
Gneiss	720-2,160	6-15	15-50	30-90	40-100	40-100
Schist	145-720	1-6	4-15	6-30	8-40	12-55

Notes:

1. The allowable bearing values are applicable for rock core having recoveries of 90 percent or greater. Lower recoveries could indicate the presence of clay seams, very weak rock zones or voids which may control the bearing capacity of the rock mass. If the recovery is less than 90 percent, lower bearing values shall be used to reflect these conditions.
2. The range of allowable bearing values corresponds to the range in uniaxial compressive strength of the rock, with lower bearing values corresponding to the lower rock strength, and higher bearing values corresponding to the higher rock strength limited to the allowable bearing stress in the concrete. The bearing values were developed by assuming a rock mass cohesion, c , equal to 10 percent of C_o , a rock mass angle of internal friction, ϕ_{rms} , equal to zero, and a factor of safety of 2.5.
3. The allowable bearing values shall be determined based on the condition of material within 2B below isolated rectangular footings and within 4B below strip footings (i.e., $L > 5B$). Where rock strata within these zones of influence of footings are variable, the material having the weakest capacity will control the allowable bearing pressure of footings.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 11 – ABUTMENTS, PIERS AND WALLS

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11.1 SCOPE

The following shall replace A11.1:

Provisions of this section shall apply for the design of abutments, piers and retaining walls. Retaining wall types addressed include:

- Rigid gravity and semi-gravity walls
- Anchored walls
- Mechanically-stabilized earth (MSE) walls
- Prefabricated modular walls
- Nongravity cantilevered walls

Abutments, piers and retaining walls shall be designed for all applicable loads, including, but not limited to, lateral earth and water pressures, including any live and dead load surcharge, impact loads, the self weight of the wall and temperature and shrinkage effects.

Retaining walls shall be designed considering the potential long-term effects of corrosion, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the wall. Retaining walls are considered permanent if the service life is more than 36 months. Permanent retaining walls shall be designed for a minimum service life of 100 years to retain an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life. Retaining walls for temporary applications are designed for a service life of 36 months or less.

11.1.1P Use

Selection of wall type shall be based on an assessment of the magnitude and direction of loading, depth to suitable foundation support, presence of deleterious environmental factors, proximity of physical constraints, tolerable and differential settlements, facing appearance, and ease and cost of construction and maintenance.

Only approved MSE and prefabricated modular wall systems shall be used. See Bulletin 15 and Appendix K for approved wall systems.

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11.1.1.1P Abutments*11.1.1.1.1P Stub Abutment*

Stub abutments are located at or near the top of approach fills, with a backwall depth sufficient to accommodate the structure depth and bearings which sit on the bearing seat.

In general, a stub abutment without a slope wall shall have a minimum 4 ft. wide bench in the fill immediately in front of the abutment. At locations where a slope wall is desirable, the bench shall be omitted, and construction of the slope wall shall conform to the requirements of the Department's standard specifications and standard drawings.

See Figure 11.1.1.1.1P-1 for typical bench details.

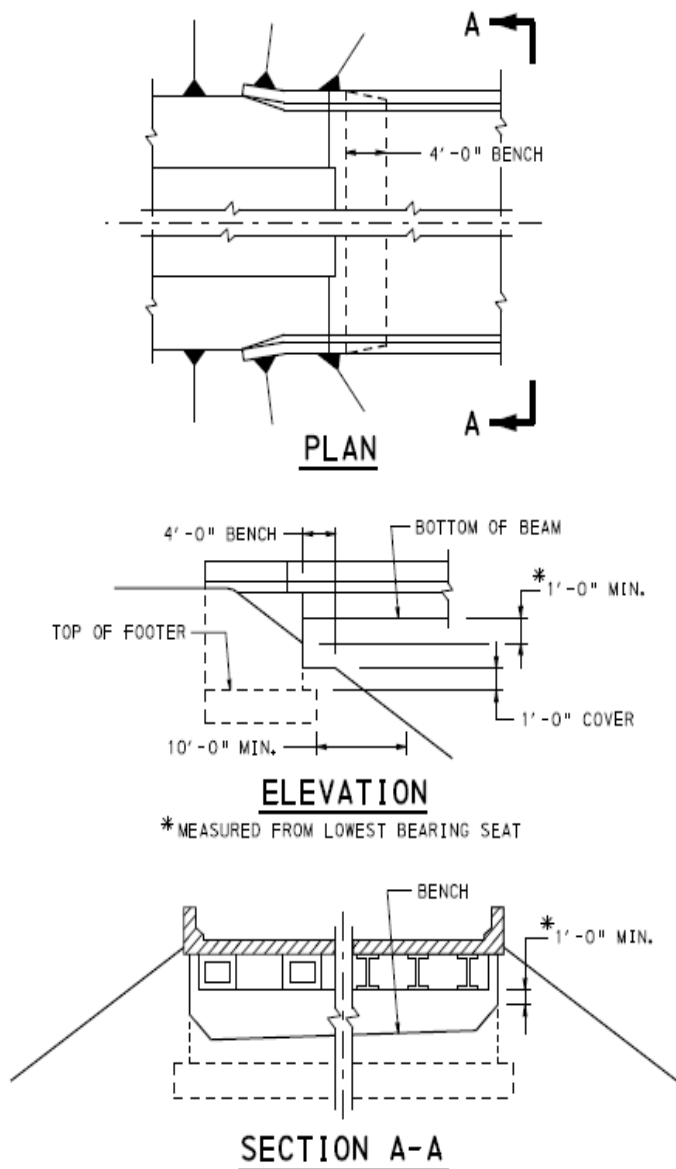


Figure 11.1.1.1.1P-1 - Recommended Details for Stub Abutments

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11.1.1.1.2P Partial Depth Abutment

Partial depth abutments are located approximately at mid-depth of the front slope of the approach embankment. The higher backwall and wingwalls may retain fill material, or the embankment slope may continue behind the backwall. In the latter case, a structural approach slab or end span design must bridge the space over the fill slope, and curtain walls are provided to close off the open area. Inspection access should be provided for this situation.

11.1.1.1.3P Full-Depth Abutment

Full-depth abutments are located at the approximate front toe of the approach embankment, restricting the opening under the structure.

11.1.1.1.4P Integral Abutment

Integral abutments are rigidly attached to the superstructure and are supported on a deep foundation capable of permitting necessary horizontal movements.

11.1.1.2P Piers*11.1.1.2.1P Solid Wall Piers*

Solid wall piers are designed as columns for forces and moments acting about the weak axis and as piers for those acting about the strong axis. They may be pinned, fixed or free at the top, and are conventionally fixed at the base. Short, stubby types are often pinned at the base to eliminate the high moments which would develop due to fixity. Earlier, more massive designs were considered gravity types.

11.1.1.2.2P Double Wall Piers

More recent designs consist of double walls, spaced in the direction of traffic, to provide support at the continuous soffit of concrete box superstructure sections. These walls are integral with the superstructure and must also be designed for the superstructure moments which develop from live loads and erection conditions. The use of double wall piers shall be subject to the approval of the Chief Bridge Engineer.

11.1.1.2.3P Bent Piers

Bent-type piers consist of two or more transversely spaced columns of various solid cross-sections, and these types are designed for frame action relative to forces acting about the strong axis of the pier. They are usually fixed at the base of the pier and are either integral with the superstructure or with a pier cap at the top. The columns may be supported on a spread- or pile-supported footing, or a solid wall shaft, or they may be extensions of the piles or shaft above the ground line.

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11.1.1.2.4P Single Column Piers

Single column piers, often referred to as "T" or "Hammerhead" piers, are usually supported at the base by a spread- or pile-supported footing, and may be either integral with, or provide independent support for, the superstructure. Their cross-section may be of various shapes and the column can be prismatic or flared to form the pier cap or to blend with the sectional configuration of the superstructure cross-section. This type pier can avoid the complexities of skewed supports if integrally framed into the superstructure and their appearance reduces the massiveness often associated with superstructures.

11.1.1.2.5P Tubular Piers

The configuration may be as described in D11.1.1.2.1P through D11.1.1.2.4P. Because of their vulnerability to lateral loadings, tubular piers shall be of sufficient wall thickness to sustain the forces and moments for all loading situations as are appropriate. Prismatic configurations may be sectionally precast or prestressed as erected. Tubular piers of hollow core section may be of steel, reinforced concrete or prestressed concrete, of such cross-section to support the forces and moments acting on the elements. The use of tubular piers shall be subject to the approval of the Chief Bridge Engineer.

11.1.1.3P Rigid Gravity and Semi-Gravity Walls

Rigid retaining walls of stone masonry or concrete construction may be used for bridge substructures to support earth slopes adjacent to roadways, or for grade separations. Rigid retaining walls include gravity, semi-gravity, cantilevered and counterfort walls. These wall types shall be used for permanent applications.

11.1.1.4P Anchored Walls

Anchored walls are applicable for temporary and permanent support of stable and unstable soil and rock masses. Anchors are usually required for support of both temporary and permanent nongravity cantilevered walls higher than about 15 ft., depending on soil conditions, and for stabilizing gravity structures, where applicable.

Anchored walls are typically constructed in cut situations in which construction occurs from the top down to the base of

C11.1.1.2.5P

When reinforced concrete tubular piers are subjected to seismic loading, the pier stem may not provide adequate confinement of the vertical reinforcing steel.

C11.1.1.4P

Anchored walls may be used for widening roads or construction grade separations in stable ground masses and for stabilizing landslides in unstable ground masses.

Anchored walls are not applicable for all ground conditions, and consideration should be given to limitations when evaluating the feasibility of using an anchored wall for a particular site. In general, poor wall performance has been associated with unsatisfactory behavior of anchors installed in caving materials (if casing is not used), organic soils, and cohesive soils with a Plasticity Index greater than 20%, which may be susceptible to creep. The feasibility of using anchors bonded in creep-sensitive soils should be evaluated in a precontract test program. Anchors should not be bonded in organic soils.

Anchored walls have been used to support earth fills. However, because of the difficulties which arise in

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the wall. Anchored walls in fill situations require the approval of the Chief Bridge Engineer.

Anchors may be prestressed ground anchors or dead-man-type elements comprised of tendons or bars extending from the wall face to a grouted zone or mechanical anchorage located beyond the zone of soil applying load to the wall.

Anchors can be installed in existing rigid gravity and semi-gravity retaining walls to provide additional resistance to sliding and/or overturning.

11.1.1.5P Mechanically Stabilized Earth Walls

MSE walls may be used where conventional gravity, cantilever, or counterforted concrete retaining walls are feasible. These walls are particularly well suited where substantial total and differential settlements are anticipated. The allowable settlement of MSE walls is limited by the longitudinal deformability of the facing and the ultimate purpose of the structure.

The proprietary mechanically stabilized earth wall systems which are approved for use, subject to the design requirements and limitations contained in this manual, are given in Appendix K.

When constructed on fills, the embankment between original ground and the leveling pad shall be composed of a granular fill in conformance with Publication 408, Section 206.2(a)1.1.b Granular Material, 206.2(a)1.1.d Rock, or 206.2(b) Select Granular Material (2RC).

The backfill requirements must be per Standard Special Provision c80201.

11.1.1.6P Prefabricated Modular Walls

Prefabricated modular wall systems, whose elements may be proprietary, generally employ interlocking soil-filled reinforced concrete or steel modules or bins, which resist earth pressures by acting as gravity retaining walls. Prefabricated modular systems may be used where conventional gravity, cantilever or counterfort concrete retaining walls are considered.

The proprietary precast modular systems which are approved for use, subject to the design requirements and limitations contained in this manual, are given in Appendix K.

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constructing anchored walls to support earth fills, their use is restricted and normally other wall types are used. The design of anchored walls for fill application must include provisions for protection of the anchors from damage during backfill operations or due to subsequent backfill and foundation soil settlements. This can be accomplished by providing ungrouted protective casing or pipe around the anchors through their unbounded length to isolate the anchors from the surrounding soil.

To prevent excessive lateral wall deflections due to anchor stressing prior to the completion of backfill placement (i.e., when the backfill does not provide sufficient passive resistance to prevent undesirable wall deflections), grouted casing or struts can be used between the wall and natural ground or rock as a reaction mechanism to prevent distress.

C11.1.1.5P

All designs must be based on equal, sound and compatible design principles. MSE walls are subject to the same external stability design criteria as conventional retaining walls, independent of the type of reinforcing system utilized. The structure must be stable with respect to sliding, overturning, foundation bearing failure and overall slope stability.

In spite of some unique design features associated with each proprietary system, several design similarities exist among all systems. A mechanically stabilized retaining wall system has three major components: reinforcements, backfill and facing elements. The reinforcement is described by the type of material used and the reinforcement geometry. The backfill used within the reinforced zone is granular to meet stress transfer, durability and drainage requirements. Facing elements are provided to retain fill material at the face. Typical facing elements include precast concrete panels with or without architectural treatments.

C11.1.1.6P

Concrete modular systems are designed as conventional gravity retaining walls developing lateral resistance primarily from self weight. In general, the full weight of the soil in the module cannot be relied upon in estimating resistance against overturning. The portion of soil weight effective in resisting overturning is limited to that weight which can be transferred in friction to the surrounding modules. Concrete modular retaining walls approximate the elements of solid gravity walls in their usual proportions, and satisfy Coulomb's lateral earth pressure criteria which require that the failure wedge be

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Metallic modular systems shall not be used.

The backfill requirements must be per Standard Special Provision c80221.

11.1.1.7P Nongravity Cantilevered Walls

Nongravity cantilevered walls may be used for the same applications as rigid gravity and semi-gravity walls, as well as temporary or permanent support of earth slopes, excavations, or unstable soil and rock masses. Nongravity cantilevered walls are generally limited to a maximum height of 15 ft., unless they are provided with additional support by means of anchors. For these cases, a complete structural analysis is required. Use of permanent nongravity cantilevered walls in fill situations requires the approval of the Chief Bridge Engineer.

Permanent nongravity cantilevered walls may be constructed of reinforced concrete and/or metals. Temporary nongravity cantilevered walls may be constructed of reinforced concrete, metal and/or timber. Suitable metals generally include steel and galvanized steel for components, such as piles, anchor head assemblies, brackets and plates, lagging and concrete reinforcement.

11.1.1.8P Temporary Excavation Support for Structures

The following are some guidelines for preparing the contract documents for projects that will require temporary excavation support for structures.

- The engineer should verify that shoring is required and that a shoring system can be designed and constructed at the locations indicated.
- The engineer should show the approximate locations of the shoring with a simple line diagram (i.e. do not show as sheet piling). Do not show stations and offsets for the wall limits, lengths or heights or other dimensions regarding the wall. The plans must show constraints that the contractor must follow in designing and detailing the wall such as R.O.W., horizontal clearances to highway or railroad, etc.
- The engineer is to provide the applicable soil properties in the standard special provision “TEMPORARY EXCAVATION AND SUPPORT PROTECTION SYSTEM.” For soil properties that are not applicable indicate as “Not Applicable”.
- The contractor is responsible for the design, limits, location, structure type, details, etc. (essentially this is a design-build item). The contractors should be given the freedom to install the most economical shoring system based on safe and economical methods and operations.

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bounded on one side by the pressure surface and on the other side by the surface of rupture.

C11.1.1.7P

Typical nongravity cantilevered walls include soldier-pile and lagging and sheet pile walls.

Stiffer wall elements, such as concrete posts or large steel beams up to 15 ft. in height and embedded properly in concrete caissons of sufficient designed diameter in very stiff subgrades such as dense sand and gravel or stiff clays or rock have been performing satisfactorily

C11.1.1.8P

To facilitate necessary construction activities such as installing formwork, the designer shall provide adequate working clearances when locating temporary support structures. See Figure C11.1.1.8P-1 for guidelines.

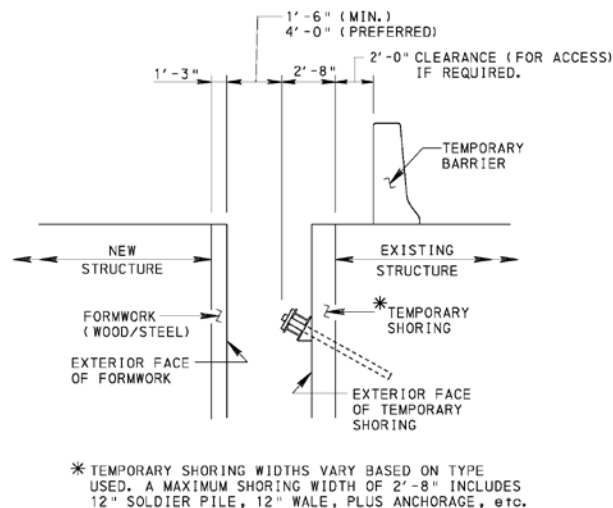


Figure C11.1.1.8P-1 – Dimensional Guidelines for Locating Temporary Support Structures

The amount of working clearance will vary by structure and should be based on the environment of the related structure(s). Common practices have established that allowing for a minimum of 1'-6", the preferred value is 4'-0", clear between temporary support construction and any adjacent

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structure/feature, is considered adequate for construction of formwork and the movement of equipment. Special circumstances may increase this clear distance.

Conventional practices for the construction of temporary support structures located near or adjacent to railroad facilities indicate clearances of no less than 12 ft. clear from centerline of tangent trackage up to 15 ft. clear from centerline for non-tangent trackage.

11.2 DEFINITIONS

The following shall replace the 4th bullet under Abutment.

Integral Abutment – Integral abutments are rigidly attached to the superstructure and are supported on deep foundations capable of permitting necessary horizontal movements.

11.4 SOIL PROPERTIES AND MATERIALS**11.4.1 General****C11.4.1**

Delete AC11.4.1.

The following shall replace A11.4.1.

Cohesionless soils with a maximum fines content of 5% by weight shall be used for backfill. This criteria can be met by AASHTO No. 57 coarse aggregate or open graded subbase (OGS) conforming to the requirements of Publication 408, Section 703.

11.4.3P Subsurface Exploration and Testing Programs

The provisions of A10.4 and D10.4 shall apply.

For major retaining walls (i.e., $H > 30$ ft.), borings shall be taken at approximately 100 ft. intervals along their alignment and at selected locations in back of the wall face to determine the nature and strength of the soils encountered. For anchored walls, borings must include sampling of soil or rock into which the anchors will be bonded.

11.4.4P Foundation Submission

Sufficient information shall be developed to comply with PP1.9.4. As a minimum, the foundation submission letter shall address the following:

- Interpretation of all subsurface and laboratory investigation results to develop foundation and earth support design parameters
- Basis for estimation of all soil/rock strengths and other design parameters
- Suitable type and depth of foundations and bearing resistance of foundation soil and rock, and method used to

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estimate ultimate bearing resistance

- Required foundation improvement(s) and recommended method(s) (e.g., extent of unsuitable material to be removed)
- Drainage requirements
- Acceptable alternative retaining wall systems
- Maximum estimated settlement during construction and service life
- Scour depth for each substructure unit at stream crossings
- Presence of corrosive soil or groundwater conditions

11.5 LIMIT STATES AND RESISTANCE FACTORS**11.5.1 General**

The following shall supplement A11.5.1.

The design service life for retaining walls, abutments and piers is 100 years.

11.5.2 Service Limit States

The following shall replace A11.5.2.

Abutments, piers and walls shall be investigated for excessive displacement at the service limit state in accordance with A10.6.2, A10.7.2, A10.8.2, A10.9.2, A11.10.4, A11.11.3, D10.6.2, D10.7.2, D10.9.2, D11.6.2, and D11.9.3. For design, the horizontal movement at the top of abutment footings shall not exceed 1/2 in..

11.5.4 Resistance Requirement

The following shall replace the first paragraph of A11.5.4.

Abutments, piers, retaining structures, their foundations and other supporting elements shall be proportioned, as specified in A11.6, A11.7, A11.8, A11.9, A11.10, A11.11, D11.6, D11.7, D11.8, D11.9, D11.10, D11.11 and D11.13P, so that their resistance satisfies A11.5.5.

11.5.6 Resistance Factors

The following shall replace Table 11.5.6-1 in A11.5.6.

Abutments, piers, retaining structures, their foundations and other supporting elements shall be proportioned, as specified in A11.6, A11.7, A11.8, A11.9, A11.10, A11.11, D11.6, D11.7, D11.8, D11.9, D11.10, D11.11, and D11.13P, so that their resistance satisfies A11.5.5.

C11.5.2

Delete the last sentence of the 3rd paragraph of AC11.5.2.

The following shall supplement AC11.5.2.

A MSE wall with welded wire or geosynthetic facing is not permitted for permanent walls.

C11.5.4

The following shall replace AC11.5.4.

Procedures for calculating nominal resistance are provided in A11.6, A11.7, A11.8, A11.9, A11.10, A11.11, D11.6, D11.7, D11.8, D11.9, D11.10, D11.11 and D11.13P for abutments and retaining walls, piers, nongravity cantilevered walls, anchored walls, mechanically-stabilized earth walls, prefabricated modular walls, and gabion walls.

C11.5.6P

Refer to Appendix A of LRFD, Section 11, in Barker, et al, (1991) regarding the selection of performance factors as a function of reliability.

Discrete vertical elements, such as soldier piles, should be treated as individual deep foundation elements. Continuous vertical elements, such as tangent piles and slurry trench concrete walls, should be treated as shallow strip footings.

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Table 11.5.6-1 - Resistance Factors for Retaining Walls

WALL TYPE AND CONDITION		RESISTANCE FACTOR
Abutments and Conventional Walls		
Bearing resistance		D10.5.5 applies
Sliding resistance		D10.5.5 applies
Sliding Resistance (Construction Condition Only)	Precast concrete placed on sand	
	• using ϕ_f estimated from SPT data	1.00
	• using ϕ_f estimated from CPT data	1.00
	Concrete cast-in-place on sand	
	• using ϕ_f estimated from SPT data	1.00
	• using ϕ_f estimated from CPT data	1.00
	Sliding on clay is controlled by the strength of the clay when the clay shear is less than 0.5 times the normal stress, and is controlled by the normal stress when the clay shear strength is greater than 0.5 times the normal stress (see Figure 1).	
	Clay (where shear resistance is less than 0.5 times normal pressure)	
	• using shear resistance measured in lab tests	0.95
	• using shear resistance measured in field tests	0.95
	• using shear resistance estimated from CPT data	0.90
	Clay (where the resistance is greater than 0.5 times normal pressure)	0.95
Nongravity Cantilevered and Anchored Walls⁽¹⁾		
Axial compressive resistance of vertical elements		D10.5.5 applies
Passive resistance of vertical elements	Passive resistance of vertical elements in soil or rock	0.75
Pullout resistance of anchors	Anchor pullout resistance	
	• Cohesionless Soil	
	correlation with SPT resistance – corrected for overburden pressure	0.55
	pullout load tests	0.65
	• Cohesive Soil	
	correlation with unconfined compressive strength	0.55
	using shear strength from lab tests	0.55
	using shear strength from field tests	0.55
	pullout load tests	0.55
	• Rock	
	presumptive values	0.45
	using minimum shear resistance measured in lab tests - soft rock only	0.50
	laboratory rock-grout bond tests	0.60
	pullout load tests	0.65

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WALL TYPE AND CONDITION		RESISTANCE FACTOR
Tensile resistance of anchors		A6.5.4.2 and A6.5.5 apply
Flexural resistance of vertical elements	• Reinforced Concrete (A5.5.4.2.1)	0.90
	• Steel (A6.5.4.2)	1.00
	• Timber (A8.5.2.2)	0.85
Mechanically Stabilized Earth Walls⁽¹⁾		
Bearing resistance		D10.5.5 applies
Sliding (Soil on Soil)		1.00
Tensile resistance of metallic reinforcement	Strip reinforcements	
	• Yielding of gross section less sacrificial area	0.85
	• Fracture of net section less sacrificial area	0.70
	Grid reinforcements	
	• Yield of gross section less sacrificial area	0.75
	• Fracture of net section less sacrificial area	0.60
	Connectors	
	• Yielding of gross section less sacrificial area	0.75
	• Fracture of net section less sacrificial area	0.60
Ultimate pullout resistance		0.9
Prefabricated Modular Walls⁽¹⁾		
Bearing		D10.5.5 applies
Sliding (Concrete on Soil or Rock)		D10.5.5 applies
Sliding (Soil on Soil)		1.00
Passive resistance		D10.5.5 applies

(1) Proprietary wall types or wall components on conventional wall types must be approved in accordance with PP1.14 "System Approval" prior to incorporation into any project.

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**11.6 ABUTMENTS AND CONVENTIONAL
RETAINING WALLS****11.6.1 General Considerations****11.6.1.2 Loading**

The following shall replace the first paragraph of A11.6.1.2.

Abutments and retaining walls shall be investigated for all applicable load combinations from A3.4.1, including, as a minimum:

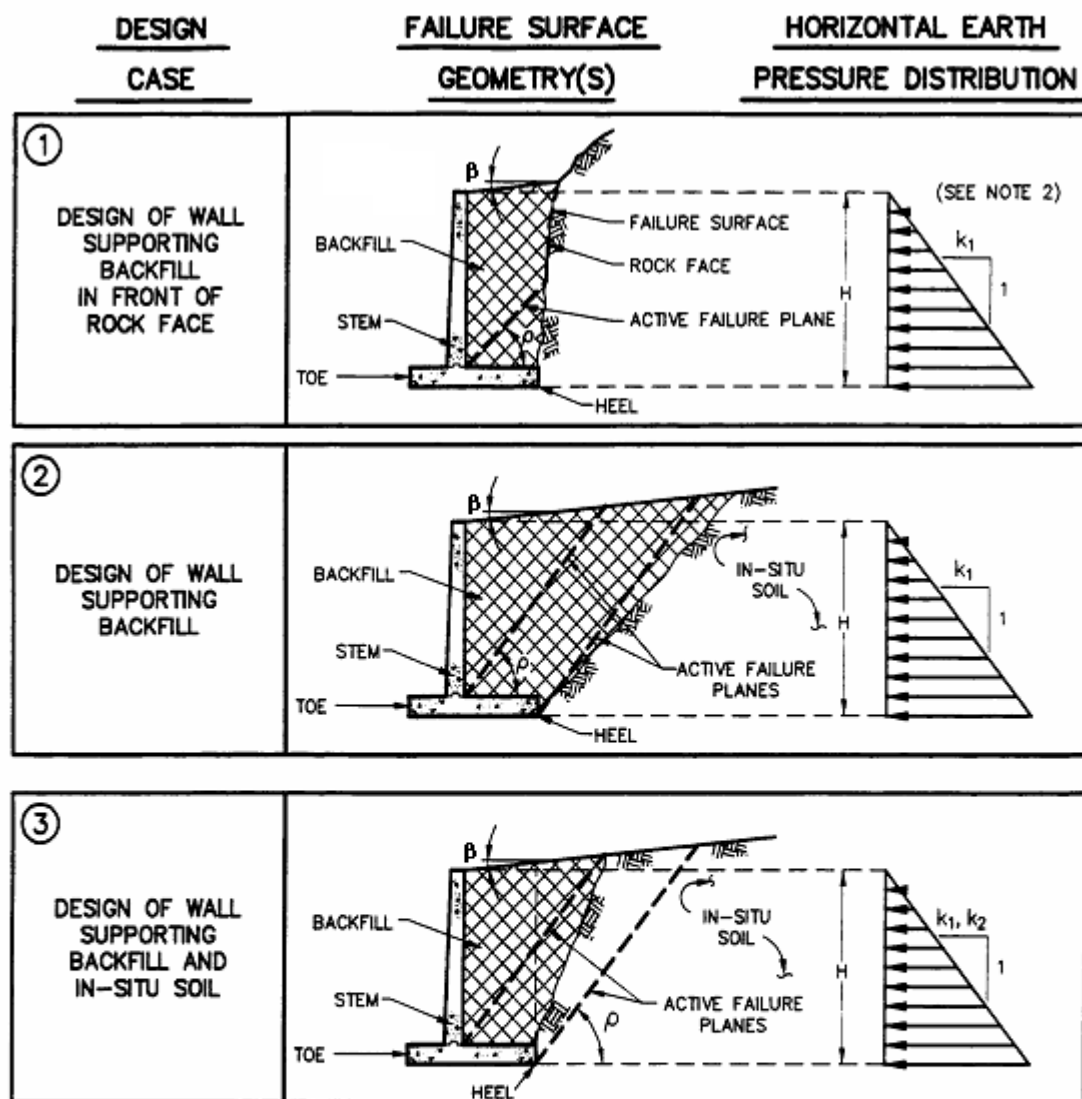
- lateral earth and water pressures, including any live and dead load surcharge
- the self weight of the wall
- temperature and shrinkage deformation effects

The following shall supplement A11.6.1.2.

The magnitude of lateral earth pressure appropriate for evaluating safety against structural and soil failure is controlled by the geometry of the soil backfill as illustrated in Figure 11.6.1.2P-1. In some instances (e.g., Case 3 in Figure 11.6.1.2P-1), structural design of the stem is controlled by the backfill, whereas evaluation of soil failure is controlled by the in-situ soil. Where a wall is located in front of a rock face (e.g., Case 1 in Figure 11.6.1.2P-1), the magnitude of lateral earth pressure may be less than that determined using the methods in A3.11.5 and D3.11.5. For such cases, Culmann's graphical procedure may be used to determine the magnitude and location of the lateral earth pressure resultant (Terzaghi and Peck, 1967).

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**LEGEND** k_1 = UNIT HORIZONTAL SOIL PRESSURE DUE TO BACKFILL k_2 = UNIT HORIZONTAL SOIL PRESSURE DUE TO IN-SITU SOIL ρ = ANGLE BETWEEN ACTIVE FAILURE PLANE AND HORIZONTAL ϕ'_t = WEIGHTED AVERAGE EFFECTIVE STRESS ANGLE OF INTERNAL FRICTION ALONG FAILURE PLANE $\rho \approx (45^\circ + \phi'_t/2) - \beta$ (SEE HUNT, 1986)**NOTES:**

(1) DETERMINE EARTH PRESSURE IN ACCORDANCE WITH A3.11.5.

(2) THE EARTH PRESSURE RESULTANT FOR THIS CONDITION CAN BE MORE ACCURATELY DETERMINED BY CULMANN'S GRAPHICAL CONSTRUCTION [SEE TERZAGHI AND PECK (1967)].

(3) ADD PRESSURES DUE TO WATER AND SURCHARGE (INCLUDING LIVE LOAD SURCHARGE) IN ACCORDANCE WITH A3.11.3 AND A3.11.6, RESPECTIVELY.

Figure 11.6.1.2P-1 - Failure Surface and Horizontal Earth Pressure Distribution for Cantilever Retaining Walls

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For temporary (construction) conditions, wind load on the abutment may be neglected, and loads considered shall include the following:

- Dead load of abutment up to bridge seat elevation
- Dead load of backfill up to bridge seat elevation
- Lateral earth pressure and live load surcharge

Walls shall be designed for a minimum live load surcharge (LS) equal to 3 ft. of soil, or the actual surcharge determined in accordance with A3.11.6.4, whichever is greater.

11.6.1.3 Integral Abutments

The following shall supplement A11.6.1.3.

The design of integral abutments, including applicable limits is to be in accordance with D11.6.4.6P. Construction specifications for integral abutments shall preclude placement of backfill until the superstructure is erected and connected to the abutments.

For additional information, refer to Appendix G.

11.6.1.5 Reinforcement*11.6.1.5.2 Wingwalls*

Delete A11.6.1.5.2

A keyed joint is not required between the wingwall and abutment wall per the Standard Drawings.

11.6.1.6 Expansion and Contraction Joints

The following shall replace A11.6.1.6

Construction joints shall be provided at intervals not exceeding 30 ft. and expansion joints at intervals not exceeding 90 ft. for gravity or reinforced concrete walls. All joints shall be filled with approved filling material to ensure the function of the joint. Joints in abutments shall be located approximately midway between the longitudinal members bearing on the abutments.

11.6.1.7P Backwalls and End Diaphragms

Backwalls shall be provided as follows:

- For each abutment where the girder depth exceeds 6 ft.
- For the fixed end abutment if the top of deck movement caused by rotation due to live and impact loads exceeds 0.5 in., or

C11.6.1.6P

An exception may be made for BRADD generated drawings, where construction joints shall be provided at intervals not exceeding 45 ft..

C1.6.1.7P

A 0.5 in. (maximum) crack between the full-depth diaphragm and approach flexible pavement or approach slab is easily maintainable if there is a crack sealing program.

Generally, a 6 ft. girder depth (approximately 7 ft. total superstructure depth) with a 30-degree skew will result in an approximate 0.5 in. movement at the fixed end. Similarly, a 90 ft. steel span with a 5 ft. superstructure depth and a 30° skew will result in an approximate 0.5 in. movement at the expansion end. For a prestressed concrete bridge, an

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- For the expansion end abutment if the top of deck movement caused by temperature change relative to fall only from 68° F, plus rotation due to live and impact loads, exceeds 0.5 in.

Provide full-depth concrete end diaphragms where backwalls are not required.

Deck deflections shall be estimated in accordance with D2.5.2.6.2.

For structures with a joint (i.e. strip seal dam or tooth expansion dam) between the deck slab and backwall, the construction specifications shall require that deck slab concrete be placed up to the blockout limits prior to construction of the full-depth concrete end diaphragms or top of the backwall to permit compensation for the beam end rotation caused by the weight of the deck.

Cast full-depth concrete end diaphragms a minimum of two hours prior to the placement of the portion of deck slab above the end diaphragm when the diaphragm is greater than 3 feet in depth.

11.6.2 Movement at the Service Limit State

The following shall replace A11.6.2.1 and A11.6.2.2.

Criteria for tolerable movement of abutments and retaining walls shall be developed based on the function and type of wall, anticipated service life and consequences of unacceptable movements. Vertical displacements shall be estimated in accordance with the provisions of A10.6.2.4, A10.7.2.3, A10.8.2.2, and D10.6.2.4 as applicable, and the maximum vertical displacement shall be limited to 1.0 in.. Horizontal displacements at the top of the foundation shall be estimated in accordance with the provisions of A10.7.2.4, A10.8.2.3, and D10.7.2.4 as applicable, and the maximum horizontal displacement shall be limited to 0.5 in.. Tilting or translation of walls above the foundation level may be estimated using information provided in AC3.11.1.

11.6.2.3 Overall Stability

The following shall supplement A11.6.2.3.

The provisions of D10.6.2.5 shall apply.

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approximate 0.5 in. movement at the expansion end will control for a bridge having a superstructure depth of 7 ft., span of 150 ft., and skew of 30°.

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11.6.3 Bearing Resistance and Stability at the Strength Limit State**11.6.3.1 General**

The following shall replace A11.6.3.1.

Abutments and retaining walls shall be proportioned to ensure stability against bearing capacity failure, overturning and sliding. Where a wall is supported by clayey foundation, safety against deep-seated foundation failure shall also be investigated.

When considering temporary conditions during construction (i.e., backfill placed prior to superstructure erection), cantilevered abutments may be evaluated for stability against overturning and sliding failure using the reduced loadings and increased resistance factors defined in D11.5.6, D11.6.1.2, D11.6.3.3, and D11.6.3.6.

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C11.6.3.1P

Common dimensions for conventional retaining walls are shown in Figure C11.6.3.1P-1.

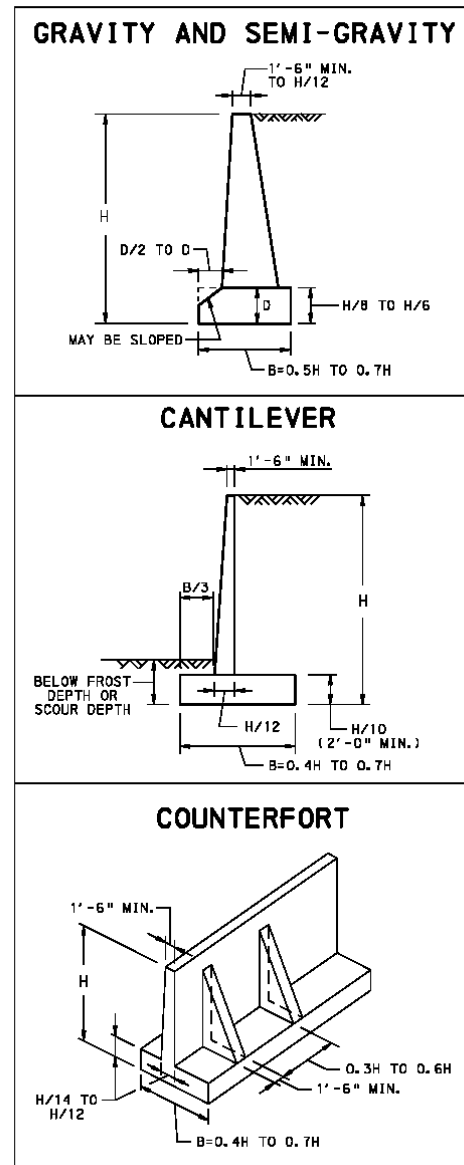


Figure C11.6.3.1P-1 - Common Dimensions for Conventional Retaining Walls (after Hunt, 1986)

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COMMENTARY

11.6.3.2 Bearing Resistance

The following shall replace A11.6.3.2.

Bearing resistance shall be investigated at the strength limit state, assuming the following soil pressure distributions:

- if the wall is supported by a soil foundation, or rock modeled as soil:

a uniformly distributed pressure over the effective base area, as shown in Figure 11.6.3.2-1.

- if the wall is supported by a hard rock foundation:

a linearly varying distribution of pressure over the effective base area, as shown in Figure D10.6.5-1P.

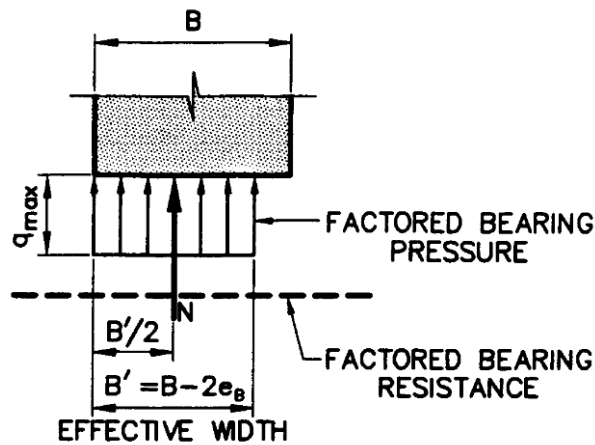


Figure 11.6.3.2-1 - Bearing Resistance Criteria for Walls with Granular Backfills and Foundations on Soil and Soft Rock, Modified after Duncan, et al, (1990)

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11.6.3.3 Overturning

The following shall supplement A11.6.3.3.

For evaluation of temporary conditions during construction of cantilevered abutments on soil, the location of the resultant of reaction forces shall be within the middle two-thirds of the base.

11.6.3.4 Subsurface Erosion

The following shall replace A11.6.3.4.

Refer to PP7.2 for scour requirements.

For cases where differential water levels may occur in front of and behind a wall, the potential for piping shall be evaluated. The hydraulic gradient for such cases shall not exceed the following (Harr, 1962):

- Fine sand and nonplastic silt (SP, SM, ML) 0.15
- Medium to coarse sand (SP) 0.20
- Sand and gravel (GW, SW) 0.30

11.6.3.5 Passive Resistance

COMMENTARY

C11.6.3.3

The following shall supplement AC11.6.3.3

Plastic deformation of foundation soils at the strength limit state results in a redistribution of the contact stress to a more uniform bearing pressure.

Base pressure resultants are maintained within the specified limits to provide the maximum possible effective foundation bearing area and minimum bearing pressures. For resultants outside of the middle third of the base for unfactored loads, a portion of the base is separated from the foundation soil or rock and is not effective in bearing. The base pressure is then redistributed over the remaining area of the base, which is in compression, resulting in higher bearing pressures and possibly tensile stresses in the concrete. The indicated limits reflect that, for identical unfactored loads, the resultant eccentricity will increase when the loads are factored.

C11.6.3.4

The following shall supplement AC11.6.3.4

The use of protective filters to protect against piping failure of soils includes use of geotextile materials in conformance to Publication 408, Section 212.

C11.6.3.5

The following shall supplement AC11.6.3.5.

The earth pressure on the back of a retaining wall will generally be active or at rest. Wall movements or rotations required to develop passive resistance in front of the wall are typically much larger (as much as 20 times larger) than movements required to develop active earth pressure behind the wall. Furthermore, the foundation for a rigid wall will usually be shallow (just deep enough to protect against frost heave and/or scour) and potentially subject to future exposure by utility excavations, road reconstruction, or other activities. Because full mobilization of passive resistance is unlikely and because soil in front of the wall may be disturbed sometime after construction, the passive resistance of soil in front of a rigid wall is neglected, unless the foundation is unusually deep or a structural foundation key is provided.

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11.6.3.6 Sliding

The following shall supplement A11.6.3.6.

For evaluation of temporary conditions during construction of cantilevered abutments, the resistance factors in Table D11.5.6-1 shall apply.

11.6.4. Safety Against Structural Failure**11.6.4.1P Base or Footing Slabs**

The rear projection or heel of base slabs shall be designed to support the entire weight of the superimposed materials, unless a more exact method is used. The base slabs of cantilever walls shall be designed as cantilever supported by the wall. The base slabs of counterforted and buttressed walls shall be designed as fixed or continuous beams of spans equal to the distance between counterforts or buttresses.

The critical sections for bending moments in footings shall be taken at the face and back of the stem. The critical sections for shear in footings shall be taken at a distance d (d =effective depth) from the face of the stem for the toe section and at the back of the stem for the heel section.

The minimum footing thickness shall be 2'-0" for footings bearing directly on soil or rock. For footings supported on piles, refer to D10.7.1.2.

11.6.4.2P Wall Stems

The upright stems of cantilever walls shall be designed as cantilevers supported at the base. The upright stems or face walls of counterfort and buttress walls shall be designed as fixed or continuous beams. The face walls (or stems) shall be securely anchored to the supporting counterforts or buttresses by means of adequate reinforcement.

Wall stems shall be designed for combined axial load (including the weight of the stem and friction due to backfill acting on the stem), and bending due to eccentric vertical loads, surcharge loads and earth pressure.

The minimum thickness of wall stems shall be 1'-6" at the top and may be of constant thickness, or with battered front and/or rear face, as required. Normally, the front face shall be vertical.

11.6.4.3P Counterforts and Buttresses

Counterforts shall be designed as T-beams. Buttresses shall be designed as rectangular beams. In connection with the main tension reinforcement of counterforts, there shall be a system of horizontal and vertical bars or stirrups to anchor the face walls and base slab to the counterfort. These stirrups shall be anchored as near to the outside faces of the face walls and as near to the bottom of the base slab as practicable.

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11.6.4.4P Reinforcement

Reinforcement requirements herein are in addition to those specified in applicable portions of D5.10, D5.11, A5.10 and A5.11.

The reinforcement in each construction panel (i.e., between vertical construction joints) of walls, with height varying uniformly from one end to another, shall be designed for the loading condition acting at one-third of the panel length from the high end of the panel. If practical, the thickness of footings shall be maintained constant in each panel or in each group of panels. The width of footings, however, may vary according to the height of wall as required by design.

Tension reinforcement at the bottom of the heel shall be provided if required during the construction stage prior to wall backfill. The adequacy of reinforcement shall be checked due to the dead load of the stem and any other vertical loads applied to the stem prior to backfilling.

11.6.4.5P Abutments

Abutment stems shall be designed for 2 in. longitudinal eccentricity from the theoretical centerline of bearing to compensate for incidental field adjustments in the locations of the bearings. The eccentricity does not need to be considered for footing design.

In general, pile-supported footings for abutments and retaining walls shall be provided with a minimum of two rows of piles. Stub abutments may be designed with one or two rows of piles, with piles battered as necessary.

Refer to criteria in D11.6.1.7P to determine whether a backwall or a full-depth concrete diaphragm wall shall be specified for an abutment. In addition to the earth, surcharge and water pressures, prescribed in D3.11.5 and A3.7, respectively, the backwalls of abutments shall be designed to resist loads due to expansion joints, and loads due to design live and impact loads. For design purposes, it shall be assumed that wheel loads are positioned so as to generate the maximum tensile stresses at the back of the backwall when combined with stresses caused by the backfill.

11.6.4.6P Integral Abutments

Integral abutments shall be designed to resist the forces generated by thermal movements of the superstructure against the pressure of the fill behind the abutment. Integral abutments should not be constructed on spread footings founded or keyed into rock. Movement calculations shall consider temperature, creep and long-term prestress shortening in determining potential movements of abutments.

Maximum span lengths and design considerations shall comply with the requirements of Appendix G, "Integral Abutments". For integral abutment details, refer to BD-667M Standard Drawing.

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11.6.6 Drainage**C11.6.6**

The following shall supplement AC11.6.6

Where a retaining wall is located in a stream environment, the wall will be subjected to varying stream levels. If the soil behind the wall cannot or is not permitted to drain freely (i.e., the backfill or retained soil is fine grained or the weep holes are clogged), the wall will be subjected to loading by differential water pressure.

11.6.7P Submittals

The following information shall be submitted by the designer and included with the construction documents for abutments and conventional retaining walls:

- Foundation submission, as required by D11.4.4P
- Earth pressures, water pressures and surcharge loadings (to be included with final plan submission)
- Geometric considerations, including beginning and ending wall stations, wall profile and alignment, right-of-way limits, utility locations, construction considerations, such as traffic restrictions or required construction sequences, and location of wall appurtenances, such as drainage outlets, overhead signs and lights, and traffic barriers (to be included in the plans)
- References and methods used for analysis for all appropriate loading conditions including all calculations (with applicable load and resistance factors), computer analyses, assumptions, input, and explanation of all symbols, notations, and formulas (to be included with final plan submission)
- Details, dimensions and schedules of all concrete and reinforcing steel, if applicable (to be included in the final plans)
- Limitations on backfill placement for integral abutments (to be included in the final plans)

11.7 PIERS**C11.7P**

The following shall supplement A11.7.

Piers shall be designed for 2 in. longitudinal eccentricity from the theoretical centerline of bearing to compensate for incidental field adjustments in the locations of the bearings. The eccentricity does not need to be considered for footing design.

The geotechnical design of pier foundations shall be in accordance with the provisions of A10 and D10. Design of piers supporting lateral earth loads shall incorporate

The PAPIER program carries the eccentricity throughout the substructure design, including the footing design, to ensure that a proper mathematical model where the moment, shear and axial force at the bottom of the column are equal to the moment, shear and axial force at the top of the footing is provided.

VBent is capable of including or neglecting the effect of the longitudinal eccentricity in the footing design. The default for PA users is to include the longitudinal eccentricity effects

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applicable provisions of A11.6 and D11.6. The foundation submission shall conform to the requirements of D11.4.4P.

11.7.0P General

Piers shall be of the open-bent (columns with cantilevered cap beams) or hammerhead-type, except at stream crossings where hammerhead or solid-section piers with rounded or protected ends shall be used. For stream crossings, the rounded or protected ends of the piers should be a minimum of 1 ft. above design flood elevation. For structures with piers skewed to the flow of a stream, circular pier stems shall be used.

Circular stems should also be considered to reduce turbulence where scour is a problem.

For pier designs, adhere to the following criteria:

- Pier columns shall be round, square, or rectangular, with a minimum diameter or thickness of 3'-0", unless a smaller column is required for aesthetic reasons. The minimum depth increment shall be 6". Solid piers shall have a minimum thickness of 2'-0" and may be widened at the top to accommodate the bridge seat when required. Minimum footing thickness is 2'-0", unless the footing is on piles, in which case 2'-6" is the minimum thickness
- Ends of pier cap shall project beyond the sides of columns, when possible, to balance positive and negative moments in the cap.
- Cap width shall be at least 6 in. wider than the thickness or diameter of the column and may be up to 1'-0" wider than the thickness or diameter of the column. Caps which support prestressed concrete beams made truly continuous may be increased in width based upon the increase in the gap between beam ends up to a maximum of 2'-0" wider than the thickness or diameter of the column, provided the centerline of bearing falls a minimum of 3 inches within the column/shaft section. Two layers of reinforcement bars are allowed to minimize cap dimensions.
- If the height of the pedestal exceeds 1'-6", the cap shall be slanted.
- Hoops of No. 4 bars at 12 in. shall be used in round pier columns. Spiral reinforcement may be used in lieu of column/pier tie reinforcement. The spiral reinforcement requirements must be designed and shown on the contract drawings.
- Bars subject to tensile stresses in cantilevered ends of cap beams shall be provided with 90° or 180° standard hooks.

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in the footing. The user has the option to override the default.

C11.7.0P

Consideration of freeboard may allow passage of ice flows and debris. An evaluation of whether freeboard (clearance above design flood elevation) is needed should be determined on a case by case basis.

The Department prefers round columns for seismic performance considerations.

Cap width with a minimum of 6 in. wider than the column thickness or diameter is specified to avoid interference between the column reinforcement that projects from the column into the cap, and the cap reinforcement. An increased gap between end face of the beams, to develop the positive moment reinforcement bars in the cast-in-place diaphragm, requires a cap width up to 2'-0" wider than the column thickness or diameter.

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- Wide columns with long internal cross ties may need lap splices at a minimum spacing of 8 ft detailed for a few locations to allow worker access during concrete placement.
- The vertical bars in columns shall extend into the cap beam a minimum of 20 bar diameters.
- Columns of pier bents shall have individual or continuous footings, depending on economy and soil conditions. Continuous footings shall be used, unless founded on rock.
- Shrinkage keys shall not be used in pier caps to eliminate partial or total shrinkage stresses.
- No. 5 reinforcement bars at a maximum spacing of 12 in. shall be provided at bottom of caps of hammerhead piers.
- For columns of pier bents located in the sloped portion of an embankment, the earth pressure against the back of the footing and column shall be increased 100% to include the effect of the adjacent embankment. The effect of the embankment in front of the column shall be neglected. Piers located in the embankment shall be investigated for stability not considering superstructure loads.
- Pier bents with continuous footings may be analyzed using the following procedure in lieu of a more exact analysis:
 - (1) Analyze the pier bent above the footing assuming the bottom of columns to be fully fixed.
 - (2) Analyze footing continuously supported by columns and loaded by soil reactions due to loads on the pier bent.
- For hammerhead piers, all the calculated cantilever reinforcement shall be extended throughout the entire length of the cap. Additional No. 5 stirrups spaced at 24 in. shall be placed in the cap within the limits of the shaft. The stirrups shall be more closely spaced near the ends of the shaft than in the interior region. In deep caps, additional longitudinal bars shall be placed at intervals throughout the depth of the cap.
- Reinforce pier caps adequately to control cracking when concrete cover of 3 in. is used in high tensile zone and corrosive environment. Alternately, build the top of the pier level or with a constant slope from one end to the other and maintain 2 in. concrete cover over the top reinforcement. If it is necessary, provide pedestals for

Excessive concrete cover at pier caps for the main (top) reinforcement is sometimes included in the design without recognizing its detrimental effect. In some instances, details do not reflect the design assumptions or calculations. Excessive concrete cover is observed when beam seats are sloped parallel to the deck slope or are stepped.

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beam seats.

- Provide closely spaced horizontal bars in the bottom half of the pier cap of hammerhead piers at each face to control shrinkage cracks. The amount of the rebars provided in this area should not be less than No. 5 at 12 in., and a bar spacing greater than 12 in. should not be used.

11.7.3P Pier Design

Piers shall be designed for longitudinal and transverse superstructure loads.

For the purpose of designing piers of multi-fixed-pier bridges, the design temperature, Δ_t , shall be taken as the larger of the temperature rise and temperature fall defined in Table D3.12.2.1-1. This design temperature change shall be used for both the expansion and the contraction of the structure.

The thermal movement at any pier location shall be determined using Equation 11.7.3P-1

$$\Delta = \alpha \Delta_t L \quad (11.7.3P-1)$$

where:

Δ = design displacement at the top of the pier

Δ_t = design temperature change

L = length of superstructure between the pier under consideration and the theoretical fixed center of structure

α = coefficient of thermal expansion of the girders

The displacement Δ shall be assumed to act parallel to the longitudinal axes of the superstructure at the pier location.

The theoretical fixed center of the structure shall be determined by assuming that the abutments are free to expand.

Pier forces caused by thermal movements of the superstructure shall be calculated assuming the pier acting

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The LRFD serviceability criteria (A5.7.3.4) permits controlled cracking of concrete. The crack width is approximately proportional almost to the square of the concrete cover over the reinforcement.

Vertical cracks in the pier cap of hammerhead piers have been observed at the intersection of the cap and the stem, generally near the middle of the cap. These cracks generally occur for wide stem (stem width 10 ft. and more) piers due to shrinkage of the pier cap concrete. The hardened concrete of the stem and protruding rebars resist free shrinkage of the cap. These cracks can be controlled by providing closely spaced horizontal bars at each face in the bottom half of the pier cap of hammerhead piers.

C11.7.3P

The design displacement represents the movement due to thermal expansion or thermal contraction. Thus, for the purpose of analysis, it may be taken in either direction such that the total forces in the pier are maximized.

The theoretical center of the structure is the point that does not move when the structure is subjected to uniform temperature change. Only one such point exists in a jointless superstructure. For other bridges, one such point exists in each segment of the structure between two expansion joints or between an expansion joint and the end of the bridge. For each segment of the structure, the sum of the thermal forces on all piers to one side of the theoretical center is equal, and opposite in direction, to the sum of the thermal forces on all piers on the other side.

Pier forces calculated using the specified reduced modulus of elasticity are one third the forces calculated assuming elastic

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elastically and assuming a concrete modulus of elasticity equal to one third the elastic modulus of elasticity.

The type of connection to the superstructure shall be considered in determining pier moments and shears.

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behavior. This reduction accounts for both the inelastic behavior of the concrete under long term deflections and the moment redistribution due to the expected rotations of the foundations.

For the strength limit state analysis, the reduction may be taken while at the same time allowing the thermal force with a 0.5 Load Factor for the conventional piers and 1.0 load factor for the integral piers of the segmental construction bridge. If a designer wants to prevent cracking at the service limit state, a Load Factor of 1.0 may be used.

Elastic pier forces are a function of the restraint of the rotation at the ends of the pier and the direction of movement relative to the major axis of the pier.

For bridges with skew angle of 90°, the elastic forces should be calculated as follows:

Piers supporting fixed bearings should be assumed to act as cantilevers. The column elastic base shear, P , and elastic base moment, M_B , are calculated as:

$$P = 3EI \frac{\Delta}{l^3} \quad (\text{C11.7.3P-1})$$

$$M_B = 3EI \frac{\Delta}{l^2} \quad (\text{C11.7.3P-2})$$

where:

Δ = design thermal movement at the pier location

E = the concrete modulus of elasticity for long term deflection

I = moment of inertia of the pier about an axis perpendicular to the longitudinal axis of the superstructure

l = the length of the column from the top of the footing or pile cap to the bottom of the bearings

For piers integral with the superstructure, the piers are assumed to act as a beam with restrained rotation (fixed) at both ends. The column elastic base shear, P , and elastic base moment, M_B , are calculated as:

$$P = 12EI \frac{\Delta}{l^3} \quad (\text{C11.7.3P-3})$$

$$M_B = 6EI \frac{\Delta}{l^2} \quad (\text{C11.7.3P-4})$$

where:

l = the length of the column from the top of the footing

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Providing a hinge at the bottom of the columns shall not be allowed.

If a column or pier bent is located in the sloped portion of an embankment, the earth pressure against the back of the footing and column shall be taken as the at rest earth pressure computed in accordance with A3.11 and D3.11. The resistance due to the passive earth pressure of the embankment in front of the column or pier bent shall be neglected.

11.8 NONGRAVITY CANTILEVERED WALLS**11.8.1 General**

The following shall supplement A11.8.1.

Hot rolled steel sheet piling may be used for permanent and temporary applications. Cold rolled steel sheet piling may be used only for temporary applications.

11.8.5 Safety Against Structural Failure

The following shall supplement A11.8.5.

The provisions of D11.9.5.2, A11.9.5.2, D11.9.5.3 and A11.9.5.3 shall apply.

11.8.5.2 Facing

or pile cap to the bottom of the superstructure

In case of skewed bridges, the components of the thermal movements should be calculated in the direction of both major axes of the pier. The appropriate moment of inertia should be used to determine the pier forces in each direction. For the component of the thermal movement in the plane of the pier, the pier should always be assumed to act as a cantilever fixed at the top of the footing or the pile cap. For the component of the thermal movements perpendicular to the plane of the pier, the pier forces will be calculated according to Equations C11.7.3P-1 through C11.7.3P-4. In this case, Δ is the component of the thermal movement perpendicular to the pier.

C11.8.1

The following shall replace AC 11.8.1.

Depending on soil conditions, nongravity cantilevered walls less than about 15 ft. in height are usually feasible.

Cold rolled steel sheet piling may be used for temporary applications, provided that the inherent strength deficiencies are adequately considered. Research has shown cold rolled sheet piling sections to have substantially less elastic moment capacity than hot rolled sections with equivalent section moduli. The difference in capacity was found to be due to the geometry of the cold rolled sections rather than the cold rolling process. The cold rolled sections were wider, deeper, and thinner than hot rolled sections with equivalent section moduli.

C11.8.5.2

The following shall supplement AC11.8.5.2.

Soil arching shall not be considered.

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11.8.7 Corrosion Protection

The following shall supplement A11.8.7.
The provisions of D11.9.7 shall apply.

11.8.8 Drainage

The following shall replace A11.8.8.
The provisions of A3.11.3 shall apply.

Seepage shall be controlled by installation of a drainage medium (e.g., preformed drainage panels, sand or gravel drains or wick drains) behind the facing with outlets at or near the base of the wall. Drainage panels shall maintain their drainage characteristics under the design earth pressures and surcharge loadings, and shall extend from the base of the wall to a level 1 ft. below the top of the wall. Only Department-approved drainage panel materials shall be specified.

C11.8.8

The following shall replace AC 11.8.8.

In general, the potential for development of hydrostatic pressures behind walls with discrete vertical elements and lagging is limited due to the presence of openings in the lagging, and the disturbance of soil behind lagging as the wall is constructed. However, the potential for leakage through the wall should not be counted upon where the ground water level exceeds one-third the height of the wall because of the potential for plugging and clogging of openings in the wall with time by migration of soil fines. It is probable that, under such conditions, a wall with continuous vertical elements (i.e., a cutoff wall) constructed with a drainage system designed to handle anticipated flows will be required.

Water pressures may be ignored in design only if positive drainage (e.g., drainage blanket, preformed drainage panels, sand drains, wick drains, etc.) with outlet pipes is provided to prevent buildup of hydrostatic pressure behind the wall.

For CIP facing rigidly attached to soldier piles, extend facing to 4 ft below ground elevation to prevent freezing of drainage blanket.

11.8.9P Submittals

The following information shall be submitted by the Designer to the Department for review and shall be shown in the construction documents for nongravity cantilevered walls:

- Foundation Submission as required by D11.4.4P
- Earth pressures, water pressures and surcharge loadings (to be included with final plan submission)
- Lateral deflection at top of wall including assessment of lateral deflection effects on adjacent features or facilities supported by the wall
- Geometric considerations including beginning and ending wall stations, wall profile and alignment, right-of-way limits, utility locations, construction considerations such as traffic restrictions or required construction sequences, and location of wall appurtenances such as drainage outlets, overhead signs and lights, and traffic barriers (to be included in the plans)
- References and methods used for analysis for all appropriate loading conditions including all calculations (with applicable load and resistance factors), computer

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analyses, assumptions, input and explanation of all symbols, notations and formulas (to be included in the plans)

- Vertical wall element types, sizes and spacings; and erection sequence (to be included with Final Plan Submission)
- Details, dimensions, connections and schedules of all structural steel and reinforcing steel for vertical wall elements and facing (to be included in the plans)
- Drainage requirements (to be included in the plans)
- Corrosion protection and/or accommodation details for the wall elements and hardware (to be included in the plans)

11.9. ANCHORED WALLS**11.9.1 General**

COMMENTARY

C11.9.1

The following shall supplement AC11.9.1.

The inclination and spacing of anchors will be dependent mostly on the soil and rock conditions, the presence of geometric constraints, and the required anchor capacity. For tremie-grouted anchors, a minimum angle of inclination of about ten degrees and a minimum overburden cover of about 15 ft. are typically required to ensure that grout fills the hole through the entire bonded length, and that adequate load transfer is achieved as a result of the confinement of the bond zone. For pressure-grouted anchors, the angle of inclination is generally not critical, and is governed primarily by geometric constraints. Also, because pressure-grouted anchors do not rely as heavily on confinement for load transfer, their bonded lengths may be located at shallower depths (e.g., 6 to 15 ft.) than is typical for tremie-grouted anchors. Very flat or steep inclination angles (ranging from 0 to 45 degrees) may be required to avoid anchorage in unsuitable soil or rock, to reach deep strata for anchorage, or to avoid underground obstructions. The minimum horizontal spacing specified between anchors (see D11.9.5.1) is intended to reduce the potential for overlap of stresses between adjacent anchors. If necessary, the inclination of adjacent anchors can be adjusted to limit the potential for stress overlap.

The minimum depth of embedment should be sufficient to provide adequate bearing resistance to support the vertical component of the anchor load(s). In addition, consideration should be given to the effects of freezing and thawing, scour, weathering and other shallow ground disturbance (e.g., utility excavations or pavement replacement) in front of or below the wall.

The following shall supplement A11.9.1.
With the approval of the Chief Bridge Engineer, anchored

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walls may be constructed in fill situations (i.e., wall construction from the bottom up) with the following restrictions:

- The full-length of all anchors extending through fill materials shall be placed within a steel casing (or strut) to minimize the effects of fill settlement on the anchors. The casing shall be of adequate size to preclude the potential for the inside top diameter of the casing from coming in direct contact with the anchor assembly as a result of settlement of the fill and/or subgrade. The casing shall extend from the back face of the wall, through the full width of backfill, and to a depth into original ground to preclude the application of differential forces on the anchor at the backfill/original ground contact. The anchor/casing assembly shall be installed by placement on the top of the compacted backfill so that the casing is in full bottom contact with the backfill.
- Solid bar anchors shall not be permitted.
- The bond length of each anchor shall extend into stable, original ground beyond the potential failure zone.
- All fill material shall consist of structural backfill and shall be compacted in conformance with Publication 408, Sections 1001.3(t) and 206.3(b), respectively. When the level of backfill is less than 1 ft. above the top of the casing or within 4 ft. of the inside face of the wall, only hand-operated compaction equipment shall be permitted.
- Stressing of any anchor level shall not proceed until backfill is placed to at least the next anchor level or to within 1 ft. of the top of the wall. To prevent transfer of load from backfill on the casing to the anchor, the unbonded length of the anchor shall not be grouted within the casing. The annulus between the casing and anchor shall be filled with grease or other approved materials for corrosion protection. The strut shall be designed to resist the required design prestress force and bending due to settlement of the fill material or live load influence.

Unprotected (uncased) anchors penetrating fill which can settle due to compression of underlying soil layers may bend as a result of settlement. Such bending is difficult to predict and would result in an unpredictable increase in the anchor load.

Solid bars are especially intolerant of bending and are prone to brittle failure.

11.9.3 Movement Under the Service Limit State

11.9.3.1 Movement

The following shall replace A11.9.3.1

The provisions of A10.6.2.4, A10.7.2, A10.8.2, D10.6.2.4, and D10.7.2 shall apply for evaluation of vertical element settlements due to the vertical component of anchor forces.

The effects of lateral wall movements on adjacent

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facilities shall be considered in the development of the design earth pressure in accordance with the provisions of A3.11.5.7 and D3.11.5.7

11.9.4 Safety Against Soil Failure**11.9.4.1 Bearing Resistance**

The following shall replace A11.9.4.1.

The provisions of A10.6.3, A10.7.3, A10.8.3, D10.6.3, D10.7.3 and D10.8.3 shall apply.

Loads at the base of vertical wall elements, including the vertical component of contributing anchor loads, shall be determined assuming that all vertical components of loads are transferred to the base of the elements. Side friction of wall elements shall not be included in the resistance to vertical loads.

11.9.4.2 Anchor Pullout Capacity

The following shall supplement A11.9.4.2.

Anchor embedment and inclination for straight shaft anchors installed in small diameter holes using low grout pressure shall follow the guidelines in Figure A11.9.1-1, unless otherwise approved by the Chief Bridge Engineer.

Final determination of the anchor pullout capacity and required bond length shall be the responsibility of the anchor wall specialty contractor.

11.9.5 Safety Against Structural Failure**11.9.5.0P General**

Anchored walls shall be designed with sufficient redundancy to protect against catastrophic wall failure in the event of the failure of an anchor.

The procedure for anchored wall design depends on the number of anchor rows and the construction sequence. For a typical wall with two or more rows of anchors constructed from the top down, the procedure requires design for the final structure with multiple rows of anchors and checking the design for the various stages of wall construction.

The required horizontal component of each anchor force shall be computed using the apparent earth pressure

C11.9.5.0P

Soil arching shall not be considered. One method of providing redundancy is to use a structural connection (e.g., bars, facing, or wales) between vertical wall elements to permit transfer and redistribution of load to adjacent anchors in the event of the failure of an anchor. Tie-back soldier pile walls with cast-in-place facing should be designed for their facings to distribute load in the event of an anchor failing. For tie-back soldier pile walls with precast facing the designer must provide a positive means to provide this required redundancy. This could be done by use of a continuous reinforced concrete cap beam or by horizontal steel tie rods run between the piles or additional anchors. Where individual soldier piles contain multiple anchors, the adjacent anchors can be designed to handle the additional load caused by this occurrence.

The procedure for design of a nongravity anchored wall depends on the number of rows of anchors utilized and the sequence of construction. For a typical wall with two or more rows of anchors constructed from the top down, the procedure requires design for multiple rows of anchors and checking the construction conditions prior to and after the installation of each row of anchors. The procedures for design or analysis of each of these cases may be as follows:

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distributions in A3.11.5.7 and D3.11.5.7, or other approved earth pressure distributions, and any other horizontal water pressure or surcharge forces acting on the wall. The total anchor force shall be determined based on the anchor inclination. The horizontal anchor spacing and anchor capacity shall be selected to provide the required total anchor force.

The vertical wall elements shall be designed to resist all applicable loads including, but not limited to, horizontal earth pressure, surcharge, water pressure and anchor loadings, as well as the vertical component of the anchor loads and any other vertical loads. Supports may be assumed at each anchor location and at the bottom of the wall if the vertical element is extended below the bottom of the wall.

The stresses in and the design of the wall facing shall be computed in accordance with the requirements of A11.9.5.3.

All components of the anchored wall system shall be designed for the various earth pressure distributions and other loading conditions which are anticipated during construction.

COMMENTARY

(a) No anchors:

Evaluate earth pressure distributions and perform analysis and design in accordance with the requirement for flexible cantilevered walls in D11.13P.

(b) One row of anchors:

- (1) Select an elevation and inclination for anchors (often dictated by physical constraints such as underground utilities).
- (2) Develop a lateral earth pressure diagram (including surcharge and water pressures) using the simplified procedures of A3.11.5.7 and D3.11.5.7 as outlined in Figures A3.11.5.7.1-1 and A3.11.5.7.2b-1. Any deviation must be approved in writing by the Chief Bridge Engineer before initiating the design. Develop an expression for the lateral pressure on the anchored portion of the vertical wall elements as a function of the anchor force, R .
- (3) Sum moments about the anchor elevation (neglecting the force R in Figure A3.11.5.7.1-1) to determine the horizontal component of the anchor force required for equilibrium and calculate the associated total anchor force based on the anchor inclination. Select anchor spacing and capacity to provide the required total anchor force.
- (4) Determine the maximum bending moments in the vertical wall elements in accordance with A11.9.5.2 and select a vertical element size and spacing (typically 6 to 10 ft.).
- (5) Design the wall facing in accordance with the requirements of A11.9.5.3.
- (6) Check the combined axial-bending capacity and bearing capacity and estimate the settlement of the vertical wall elements under the vertical component of the anchor forces and other vertical loads in accordance with A10.6, A10.7, A10.8, D10.6, D10.7 and D10.8.
- (7) Check the overall stability of the wall system, retained soil and foundation in accordance with A11.9.3.2.

For soft clays with $S_u \leq 0.3 \gamma'_s H$ (H in ft.),

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continuous vertical elements extending well below the exposed base of the wall are generally required to prevent heave in front of the wall [U. S. Steel (1984)]. Otherwise, the vertical elements are embedded as required for bearing capacity.

(Where significant embedment of the wall is required to prevent bottom heave, the lowest section of wall below the lowest row of anchors must be designed to resist the moment induced by the pressure acting between the lowest row of anchors and the base of the exposed wall, and the force $P_b = 0.7(\gamma_s H B_e - 1.4cH - \pi c B_e)$, in kip/ft acting at the mid-height of the embedded depth of the wall.).

(c) Two or more rows of anchors.

- (1) Develop an apparent lateral earth pressure diagram (including surcharge and water pressures) in accordance with A3.11.5.7 and D3.11.5.7.
- (2) Select a number of anchor level elevations based on the anticipated wall height. Vertical spacings between rows of anchors of 8 to 12 ft. are common.
- (3) Calculate the required horizontal components of the anchor forces by the proportional methods using the apparent earth pressure distribution developed under Step 1, or by other suitable methods. If the proportional method is used, the top row of anchors is assumed to support the tributary area of pressure from the top of the wall down to a point midway between the first and second anchor levels. The bottom row of anchors is assumed to support the pressure from the base of the wall up to a point midway between the two lowest rows of anchors. Alternatively, the embedded portion of the vertical wall element may be assumed to support the pressure between the base of the exposed wall and a point midway up to the lowest anchor level. The magnitude of available support may be computed in accordance with A3.11.5.9. Intermediate rows of anchors are assumed to support the pressure between the midway points to the next higher and lower rows of anchors. Calculate the total anchor force based on the anchor inclination. Select the horizontal anchor spacing and anchor capacity to provide the required total anchor force.

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- (4) Determine the maximum bending moments in the vertical wall elements in accordance with A11.9.5.2, and select a vertical element size and spacing (typically 6 to 10 ft.).
- (5) Design the wall facing in accordance with the requirements of A11.9.5.3 and D11.9.5.3.
- (6) Check the combined axial-bending capacity and bearing capacity, and estimate the settlement of the vertical wall elements under the vertical component of the anchor forces and other vertical loads in accordance with A10.6, A10.7, A10.8, D10.6, D10.7 and D10.8.
- (7) Check the overall stability of the wall system, retained soil and foundation in accordance with A11.9.3.2.

For soft clays with $S_u \leq 0.3 \gamma'_s H$ (H in ft.), continuous vertical elements extending well below the exposed base of the wall are generally required to prevent heave in front of the wall [U. S. Steel (1984)]; otherwise, the vertical elements are embedded as required for bearing capacity.

- (8) Estimate wall deflections under the design loadings and support conditions where deflections are critical.

11.9.5.1 Anchors

The following shall supplement A11.9.5.1.

The optimum vertical spacing between anchors shall be determined by minimizing the bending moment in the vertical wall elements between anchor levels with consideration of the construction sequence to be followed. Vertical spacings between anchors of 8 to 12 ft. are common.

Horizontal spacings of 6 to 10 ft. between anchors are common.

11.9.5.2 Vertical Wall Elements

The following shall supplement A11.9.5.2.

The combined axial-bending and bearing capacity, and the settlement of the vertical wall elements under the vertical component of the anchor forces and other vertical loads shall be evaluated in accordance with A10.6, A10.7, A10.8, D10.6, D10.7 and D10.8.

The overall stability of the wall system, retained soil, and foundation shall be evaluated in accordance with A11.9.4.3.

The use of welded threaded studs on soldier piles is not permitted at either the design or construction phase of a project.

C11.9.5.2

The following shall supplement AC11.9.5.2.

Structural analysis of vertical wall elements may be performed using COM624P, LPILE 5.0 or other Department approved method suitable for evaluation of laterally loaded deep foundations.

On a Department project, a tie-back wall design utilized an angle bolted on to threaded studs welded to the soldier piles in order to attach the precast panels used for its permanent facing. Several of the welds of the studs failed during installation of the precast panels. Possible alternates to this detail includes fillet welded or bolted connections of the angle to the soldier pile.

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11.9.5.3 Facing

The following shall supplement A11.9.5.3.

Do not consider temporary lagging as permanent resistance in the design of facing.

11.9.7 Corrosion Protection

The following shall replace A11.9.7.

Corrosion protection requirements will be as follows:

Prestressed anchors and anchor heads shall be protected against corrosion in a manner consistent with the conditions at the site. The level and extent of corrosion protection shall be a function of whether the anchor is intended for temporary or permanent applications of the ground environment and of the potential consequences of an anchor failure. For permanent wall applications, a minimum of double corrosion protection shall be provided, regardless of the ground environment.

Table 11.9.7-1 Criteria for Aggressive Ground Environments

PARAMETER	LIMITING VALUES
Resistivity	< 2000 ohm-cm
pH	< 5

All anchors for permanent walls shall be fully encapsulated over their entire length. For temporary walls, anchors shall be encapsulated whenever one or more of the limiting values specified in Table 11.9.7-1 is exceeded. Corrosion protection may not be required for temporary walls having a design life of less than one year. Other potentially corrosive conditions, including stray currents, shall be identified and evaluated, and appropriate means of corrosion protection shall be designed by the anchor-wall specialty contractor and shall be approved by the Department.

Corrosion protection for anchor hardware required for permanent and temporary walls shall be consistent with the level of protection required for the anchors. Structural steel elements shall be provided with additional sacrificial thickness, painted or coated to accommodate or prevent corrosion, in accordance with Department construction specifications. For structural design, sacrificial thickness shall be computed by assuming a carbon steel loss equal to 9 $\mu\text{m}/\text{year}$ after zinc coating depletion. Corrosion resistant coatings, if specified, shall be of the electrostatically applied resin-bonded epoxy type with a minimum application thickness of 15 mils in conformance with the requirements of AASHTO M 284/M 284M. For corrosion protection of concrete elements, see PP3.4.

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11.9.8 Construction and Installation**11.9.8.1 Anchor Stressing and Testing**

The following shall replace the first sentence in the first paragraph of A11.9.8.1.

All production anchors shall be subjected to load testing and stressing in accordance with the special provisions.

The following shall replace the second sentence in the second paragraph of A11.9.8.1.

The lock off load shall be in accordance with the special provisions.

11.9.9 Drainage

The following shall supplement A11.9.9.

Only Department-approved drainage panel materials shall be specified.

11.9.10P Submittals

In addition to the information in D11.8.9P for nongravity cantilevered walls, the following items shall be submitted by the Designer to the Department for review and shall be shown in the construction documents for anchored walls.

- Anchor type and estimated capacity, required capacity, minimum bonded and unbonded anchor lengths, anchor inclination, and anchor locations and spacings
- Description of anchor installation procedures including drilling and grouting
- Corrosion protection details for the anchors and anchor hardware
- Detailed plans for proof, performance, creep (if applicable), and lift-off testing of anchors including specified load measuring devices, test locations, and testing procedures
- Analysis of the stresses in vertical wall elements, facing, and anchors at critical stages of construction

11.10 MECHANICALLY STABILIZED EARTH WALLS**11.10.1 General**

The following shall supplement the first paragraph of A11.10.1.

Mechanically stabilized earth (MSE) systems, whose elements may be proprietary, should use strip or grid-type inextensible tensile reinforcements in the soil mass, and a

C11.9.8.1

The following shall supplement the last paragraph of AC11.9.8.1.

If additional strands are required for redundancy, a minimum of 40% of the ultimate tensile strength of the strands may be used.

Selection of anchor installation procedures is usually made by the anchor wall contractor. In general, however, anchors in rock, clayey soils, silts and fine sands are pressure grouted, whereas anchors in coarser grained soils are usually grouted by gravity backfilling.

C11.10.1

Delete from Figure AC11.10.1-1 the schematic Titled

- MSE Wall with CIP concrete or shotcrete facing

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discrete modular precast concrete facing which is vertical or near vertical.

The following shall supplement the third paragraph of A11.10.1.

Also, mechanically stabilized earth walls shall not be used under the following conditions:

- (a) Height greater than 40 ft. when mesh reinforcing is used and height greater than 55 ft. when strip reinforcement is used
- (b) On curves with a radius of less than 60 ft.
- (c) When longitudinal differential settlements along the face of the wall are expected to be greater than shown in Table AC11.10.4.1-1
- (d) When floodplain erosion is anticipated to undermine the reinforced fill zone, or where depth of scour cannot be reliably determined. Chief Bridge Engineer approval is required for this use
- (e) Where upstream floodplain or downstream drawdown creates unstable conditions of the backfill material and retaining wall

The following shall supplement A11.10.1.

The proprietary mechanically stabilized earth wall systems which are approved for use, subject to the design requirements and limitations contained in this manual, are given in Appendix K.

For walls supporting roadways which are de-iced with chemical additives, an impervious membrane shall be placed above the reinforced zone and sloped to a collector drain to preclude infiltration of corrosion-causing elements, as shown in Standard Drawing BC-799M.

For walls constructed in side-hill cut and fill geometries or cut, a drainage blanket shall be constructed to intercept groundwater as shown in Standard Drawing BC-799M.

Where manholes for surface drainage must be constructed within the zone of reinforcing, reinforcing elements may be skewed from their normal position perpendicular to the wall face up to a maximum skew of 25°. (Refer to Standard Drawing BC-799M). Where this is not possible, an independent wall shall be constructed in back of the drainage structure. Panels in front of such drainage structures shall be structurally connected to adjacent panels.

When constructed on fills, the embankment between the original ground and the leveling pad shall be composed of a granular material (in conformance with Publication 408, Section 206.2(a)1.1.b Granular Material, 206.2(a)1.1.d Rock, or 206.2(b) Select Granular Material (2RC)).

Typical MSE wall cut and fill sections are given on Standard Drawing BC-799M.

An allowable range of 90 to 120 pcf is permitted for unit

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weight of specified backfill. MSE wall designs must consider both the upper and lower limits of allowable backfill unit weights. This includes calculations for bearing capacity, settlement, sliding, overturning and reinforcement pullout. If a particular unit weight is desired (i.e. due to strap length restrictions for right-of-way or availability of specified backfill material), the MSE wall design may be performed for this particular weight or range of unit weights. The unit weight or range of unit weights used in the design must be noted on the drawings.

11.10.2 Structure Dimensions**11.10.2.2 Minimum Front Face Embedment**

The following shall supplement A11.10.2.2.

Consideration may be given to alternate methods of scour protection including sheetpile walls driven to below potential scour levels and/or riprap of sufficient size and placed to sufficient depth to preclude scour.

11.10.2.3 Facing

The following shall supplement A11.10.2.3.

Minimum panel reinforcement in accordance with A5.5 shall be provided. See D5.4.3.6P for corrosion protection of panel reinforcement and module reinforcement where salt spray is anticipated.

11.10.2.3.2 Flexible Wall Facing

Delete A11.10.2.3.2

11.10.4 Movement and Stability at the Service Limit State**11.10.4.1 Settlement****C11.10.4.1**

The following shall supplement AC11.10.4.1.

For MSE walls, the limiting differential settlement criteria based on joint spacing is empirically derived from observations on completed structures. Structures subjected to greater differential settlements may be damaged by concrete spalling from panel corners. Full height panels (30 ft² or more), when used, have been subject to longitudinal and vertical cracking and visible bending. Therefore, their use is not recommended without substantial revision to the design and erection procedures presently in force.

Where greater differential settlements are anticipated, a system of open vertical slip joints should be provided at suitable intervals or the foundation improved by various ground improvement techniques (such as over-excavation and replacement with compacted backfill using select material).

11.10.4.3 Overall Stability

The following shall supplement A11.10.4.3.

For structures loaded with sloping surcharges, general

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stability analyses shall be performed using Swedish circle methods and using a minimum performance factor as outlined in D10.5.

11.10.5 Safety Against Soil Failure (External Stability)**11.10.5.1 General**

The following shall supplement A11.10.5.1.

In addition, saturated soil condition must be considered in determining the external stability of the walls.

For external stability computations, live load surcharges shall be applied from a vertical plane beginning at the back of the reinforced zone.

For calculation of horizontal design forces behind the reinforced mass, consider and apply all the properties of the random backfill, which includes 1 ft. of specified backfill material.

11.10.5.2 Loading

The following shall supplement A11.10.5.2.

For external stability computations, maximum EH and EV Load Factors govern design.

11.10.5.3 Sliding

The following shall replace the first paragraph of A11.10.5.3.

The provisions of D10.6.3.4 shall apply. The vertical force due to the surcharge load shall not be used in calculating the sliding resistance given in D10.6.3.4.

11.10.6 Safety Against Structural Failure (Internal Stability)**11.10.6.2 Loading (Internal Stability)**

The following shall supplement A11.10.6.2.

For internal stability computations, maximum EH and EV Load Factors govern the design for maximum stress in the reinforcement and minimum EV for pullout computations.

11.10.6.2.1 Maximum Reinforcement Loads

The following shall supplement A11.10.6.2.1.

In Pennsylvania the Simplified Method shall be used.

11.10.6.3 Reinforcement Pullout*11.10.6.3.2 Reinforcement Pullout Design*

The following shall supplement A11.10.6.3.2.

Where pullout tests are performed, the reinforcement element configuration shall be the same as for the actual project condition. Minimum length of embedment shall be 3'-

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6" and a constant rate of strain of 1.25 in/in shall be used.

Perform specific pullout tests to determine ultimate resistance for fully saturated soil condition.

11.10.6.4 Reinforcement Strength*11.10.6.4.2 Design Life Considerations**11.10.6.4.2a Steel Reinforcement*

The following shall supplement A11.10.6.4.2a

Steel reinforcement elements in MSE walls shall be designed to have a corrosion resistance-durability to ensure a minimum design life of 100 years for permanent structures.

C11.10.6.4.2a

The following shall supplement AC11.10.6.4.2a

Where stray ground currents are anticipated within a 200 ft. distance of a structure, the potential for stray current corrosion exists when metallic reinforcements are used. Induced-current cathodic protection measures have not been successful in the past and have caused two known failures. Therefore, the use of these corrosion-mitigation systems are not recommended.

The use of alloys, such as aluminum and stainless steel, is not recommended under any circumstances. Substantial deterioration/corrosion has been observed in structures constructed with these alloys.

Delete the second sentence in the ninth paragraph of AC11.10.6.4.2a which relates to epoxy coating.

11.10.6.4.2b Geosynthetic Reinforcement

Delete A11.10.6.4.2b.

11.10.6.4.5P Redundancy

MSE walls shall be designed with sufficient redundancy to protect against catastrophic failure of each discrete facing panel in the event of the failure of one strip or one longitudinal bar per grid mesh in each discrete panel. Adequacy under this condition shall be evaluated using a load factor of 1.1 for EH and 1.0 for EV for both pullout and rupture of the soil reinforcements.

11.10.8 Drainage

The following shall supplement the second paragraph of A11.10.8.

Refer to Standard Drawing BC-799M for typical drainage blanket detail.

The following shall supplement the third paragraph of A11.10.8. Refer to Standard Drawing BC-799M for impervious membrane detail.

11.10.10 Special Loading Conditions**11.10.10.2 Traffic Loads and Barriers**

The following shall replace A11.10.10.2.

C11.10.10.2

Delete AC11.10.10.2.

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Traffic loads shall be considered in accordance with the criteria outlined in A13.

Barriers, constructed over or in line with the front face of the panel shall be designed to resist overturning moments by their own mass. Base slabs shall not have any transverse joints, except construction joints. The horizontal load shall be deemed to be transferred by horizontal shear stress to the reinforced mass and distributed to the upper row of reinforcement only. The upper row of soil reinforcement shall be sized to resist an additional load of 2 kips per linear foot of wall. The full reinforcement length shall be considered effective in resisting the impact horizontal load.

Barrier reinforcement shall be in accordance with the appropriate Standard Drawings. Other barrier reinforcement requires Chief Bridge Engineer approval. The anchoring slab shall be strong enough to resist the ultimate strength of the standard barrier.

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 3 ft. from the wall face, driven 5 ft. below grade, and spaced to miss the reinforcements. The upper two rows of reinforcement shall be designed for an additional horizontal load of 0.3 kips per linear foot of wall.

When constructed over in line with the front face of the panels or precast modules, barriers shall be designed to meet the ultimate strength of the Department's standard barrier by their own mass and ability to resist overturning moments. Base slabs shall not have any transverse joints, except construction joints. Alternately, the wall can be moved out to provide a U-wing tie of barrier from the abutment of standard shape and strength.

11.10.10.3 Hydrostatic Pressures**C11.10.10.3**

Delete AC11.10.10.3.

11.10.10.5P Design Details

The juncture of mechanically stabilized earth walls and cast-in-place structures shall be protected from loss of fines and differential settlements in accordance with the detail shown in Standard Drawing BC-799M. Dissimilar wall types shall not be constructed immediately adjacent to each other if anticipated differential settlements at the juncture are greater than 1 in. For mechanically stabilized earth walls, geotextile fabric shall be placed behind all open joints of panel facing.

11.10.11 MSE Abutments

Delete A11.10.11.

11.10.11.1P MSE Abutments on Pile Foundations

The design of bridge abutment footings and connecting backwall, supported on mechanically stabilized earth walls, shall be based on bridge loading developed by the LRFD

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method.

The MSE wall abutments (stub abutment) shall be supported on steel H-beam piles encased in smooth or corrugated galvanized steel pipe, filled with coarse sand and the MSE walls shall be designed as an earth retaining structure as shown in Standard Drawing BC-799M.

The minimum distance from the centerline of the bearing on the abutment to the outer edge of the facing shall be 3'-6". The minimum distance between the back face of the panel and the footing shall be 6 in.

To prevent runoff of potentially chemically active water from entering the reinforced soil embankment, the gap between the facing panels and abutment footing shall be sealed with an impervious liner, as shown in Standard Drawing BC-799M.

Use the following pile design guidelines:

- a. Piles should be designed for vertical loads, i.e., $(DL_{\text{Super}} + (LL + I)_{\text{Super}} + DL_{\text{stub abutment}})$
- b. Piles should be either point bearing or end bearing.
- c. Allowable bearing pressure on piles should be in accordance with foundation approval.
- d.. For construction sequences for abutments supported in piles refer to special provisions and construction specifications.
- e. In some instances, pile locations interfere with soil reinforcing grid or soil reinforcing strips behind the MSE walls. Therefore, develop specific method for the field installation to avoid interference of grids or strips, with the piles and show the details of pile locations and arrangement of MSE wall soil reinforcing elements. Cutting of reinforcing strips, steel mesh or grids at pile locations, vertical obstacles or utilities is not acceptable.
- f. Show complete drainage behind the wall as shown in Standard Drawing BC-799M as required by the field condition and also provide weep holes at 10'-0" maximum.

11.11 PREFABRICATED MODULAR WALLS**C11.11P**

Precast modular systems are sensitive to longitudinal differential settlements which may cause cracking of connecting interior members. The limiting criteria derived are empirical and based on description in the literature of bin-type wall failures.

11.11.1 General

The following shall supplement A11.11.1.

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The proprietary precast modular systems which are approved for use, subject to the design requirements and limitations contained in this manual, are given in Appendix K.

Prefabricated modular wall systems are particularly well suited in side-hill cut applications, along stream channels, and where limited space is available between the wall line and the right-of-way limits. Typically, the width of the bottom module is approximately 50% of the wall height. When constructed on fills, the embankment between the original ground and the footing shall be composed of a granular material (in conformance with Publication 408, Section 206.2(a)1.1.b Granular Material, 206.2(a)1.1.d Rock, or 206.2(b) Select Granular Material (2RC)).

Concrete modular systems shall not be used under the following conditions:

- (a) When wall heights exceed 35 ft.
- (b) For abutments where flared wingwalls are not at 30, 45, 60 or 90 degrees to the abutment wall or with open front-face modules
- (c) When calculated longitudinal differential settlements along the face of the wall are greater than 1/200

11.11.2 Loading

The following shall supplement A11.11.2.

Where the back of the prefabricated modules forms an irregular, stepped surface, the earth pressure shall be computed on a plane surface drawn from the upper back corner of the top module to the lower back heel of the bottom module using Coulomb earth pressure theory.

Maximum EH and EV Load factors govern stability computations.

Neglect the soil weight on the module beyond an average plane (apparent) surface behind the modules as shown in Figure A3.11.5.9-2. Consider the weight of the concrete modules and soils within the concrete modules even if it is beyond the average plane surface.

11.11.4 Safety Against Soil Failure**11.11.4.1 General**

The following shall replace the second paragraph of A11.11.4.1.

Passive pressures shall be neglected in stability computations.

The following shall supplement A11.11.4.1

The foundation shall be proportioned in accordance with the factored loads, bearing resistance factors and the applicable provisions of A10.6 and D10.6.

Refer to Table A3.11.5.3-1 for the coefficients of the backfill or the foundation soil.

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For structures loaded with sloping surcharges, general stability analyses shall be performed using Swedish circle methods and using a minimum resistance factor as outlined in A10.5 and D10.5.

If a computer program is submitted for the design of a proprietary wall, the submission shall include design and/or analysis methodology assumptions, and one copy of hand calculations for the most critical bottom module of the wall to demonstrate compliance of all the design requirements and the results of the computer output.

11.11.4.3 Bearing Resistance

The following shall supplement A11.11.4.3.

Alternatively, bearing pressures may be computed using a uniform base pressure distribution over an effective footing width ($B' = B - 2e_B$) in accordance with D11.6.3.2

11.11.4.5 Subsurface Erosion

The following shall replace A11.11.4.5.

Bin walls, T-walls, and any prefabricated modular walls may be used in scour-sensitive areas only where their suitability has been documented to the satisfaction of the Owner.

11.11.7 Abutments

Delete A11.11.7.

11.11.8 Drainage

The following shall supplement A11.11.8.

Refer to Standard Drawing BC-799M for drainage blanket details.

11.13P GABION RETAINING WALLS

All gabion retaining walls shall be designed based on the following guidelines:

1. General
 - a. Gabion walls may be constructed for non-structural applications, such as erosion control and slope protection against erosion for any length and height in rural areas.
 - b. As a retaining wall (structural application), gabion walls are permitted for up to 12 ft. maximum height (base to top) in rural areas only and ADT of 750 or less.

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2. Design Specifications

- a. Use gabion manufacturer's specifications for design considerations, except that load and resistance factors shall be as in D10.5.
- b. Check external stability of the gabion wall (overturning and sliding) for all applicable forces due to live load surcharge and fill slope being retained by the wall.
- c. Design the wall using Coulomb Wedge Theory and the following backfill soil characteristics.
 - Unit Weight of Backfill = 0.120 kcf
 - Horizontal Soil Pressure = 0.035 kcf

In the event that a clear interpretation of design cannot be resolved, the Chief Bridge Engineer will be the arbiter and the Chief Bridge Engineer's decision will be final.

- d. Where gabions will be exposed to corrosive environment (such as salt spray due to splashing or drainage into the wall) or industrial fumes and effluents, the basket should be constructed of galvanized and plastic coated wire.
 - e. The gabions baskets in gabion walls shall be designed to have a corrosion resistance - durability to ensure a minimum design life of 50 years
 - f. Place footing not less than 3 ft. below finished ground elevation.
 - g. Provide at regular intervals along the gabion walls a suitable drainage system (Drain Pipe) to carry water away from the foundation.
 - h. Check foundation pressure and settlement to ensure that no foundation failure occurs. District Bridge and Soil Engineers' approval should be secured before incorporating gabion walls in any project.
3. Construction Guidelines:
- Show the following notes on the plans:
- a. Provide materials and perform work in accordance with Publication 408, supplements thereto, and/or Special Provisions as indicated.

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- b. Provide geotextile fabric Class 2, Type B along all interface areas with backfill and the gabion walls. The minimum lap of fabric = 12 in..
- c. The structural backfill behind the gabion walls shall be in accordance with Standard Drawing RC-12M.

11.14P REFERENCES

The following shall supplement A11.12.

Barker, R. M., Duncan, J. M., Rojiani, K. B., Ooi, P.S.K., Tan, C. K., and Kim, S. G., "Load Factor Design Criteria for Highway Structure Foundations", Final Report NCHRP 24-4, National Research Council, Washington, DC, 292 pp, 1991

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Harr, M. E., "Groundwater and Seepage", McGraw-Hill Book Company, New York, New York, 315 pp, 1962

Hunt, R. E., "Geotechnical Engineering Techniques and Practices", McGraw-Hill, Inc., New York, New York, 729 pp, 1986

Terzaghi, K., and Peck, R. B., "Soil Mechanics in Engineering Practice", 2nd ed., John Wiley & Sons, New York, New York, 729 pp, 1967

U. S. Steel, Steel Sheet Piling Design Manual, updated and reprinted by the U. S. Department of Transportation, Federal Highway Administration, Washington, DC, 132 pp, 1984

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 12 – BURIED STRUCTURES AND TUNNEL LINERS

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12.1 SCOPE

The following shall supplement A12.1.

Culverts or buried structures refer to flexible (corrugated metal and thermoplastic) and rigid (concrete) structures used for conveyance of water or traffic below earth embankments. The structures for which the design procedures in this section are applicable range in span from 4 to 40 ft. Although pipe sizes are included herein for design consideration, pipes less than 8 ft. in diameter are generally a roadway item (see Design Manual, Part 2, Chapter 10).

12.2 DEFINITIONS

The following shall supplement A12.2.

Tied Arch - An arch section designed and constructed with an integral base slab.

12.3 NOTATION

The following shall supplement A12.3.

BD	=	load factor for dead load (dim)
BL	=	load factor for live load (dim)
d	=	depth of corrugation (in.) (D12.6.2.1)
D _h	=	horizontal span of metal pipe (in.) (D12.6.2.1)
DL	=	dead load (kips)
D _L	=	deflection lag factor (dim)
D _v	=	vertical rise of metal pipe (in.) (D12.6.2.1)
i	=	interest rate (dim)
I	=	inflation rate (dim)
K	=	bedding constant (dim)
LCC	=	present worth life cycle cost (\$)
LL	=	live load (kips)
MC	=	maintenance cost (\$)
n	=	service life (yr)
n _p	=	material design life (yr)
N _f	=	flexibility number (dim)

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P	=	design load (ksi)
PC	=	present cost (\$)
P_{DL}	=	design dead load (kip/in)
P_{LL}	=	design live load (kip/in)
PS	=	pipe stiffness (kip/in/in)
S	=	mean pipe diameter = $2 (C + ID/2)$
SV	=	salvage value (\$)
W_c	=	vertical load per unit of pipe length (kip/in)
δ_c	=	vertical crown deflection with respect to the invert (in.) (D12.6.2.1)
ϵ_b	=	bending strain in thermoplastic pipe (in/in)
ϵ_c	=	hoop strain in thermoplastic pipe (in/in)
ϵ_T	=	total strain in thermoplastic pipe (in/in)
μ_m	=	corrugated metal pipe shape factor (dim) (D12.6.2.1)

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12.4 SOIL AND MATERIAL PROPERTIES**12.4.1 Determination of Soil Properties****12.4.1.1 General**

The following shall supplement A12.4.1.1.

The provisions of A10.4.1 shall apply.

The foundation materials shall be sampled and tested to a depth below the anticipated foundation level approximately equal to the rise plus the height of cover, unless rock is encountered at a shallower depth. Where soft, compressible deposits are encountered, all exploration shall extend to a minimum depth below the anticipated foundation level equal to twice the rise, and at least one boring shall extend a minimum depth below the anticipated foundation level equal to twice the rise plus the depth of cover. The maximum drilling depth should be 50 ft., except for high fills (i.e., fill heights exceeding 50 ft.), for which the maximum drilling depth should be 50 ft. plus two times the rise.

12.4.1.3 Envelope Backfill Soils

The following shall replace A12.4.1.3.

The type, compacted density and strength properties of the soil envelope immediately adjacent to the pipe shall be established. The structural backfill shall conform to the requirements of Standard Drawings RC-12M and RC-30M and Publication 408.

12.4.2 Materials**12.4.2.2 Concrete**

The following shall replace A12.4.2.2.

For concrete requirements, see D5.4.2.1.

12.4.2.3 Precast Concrete Pipe

The following shall replace A12.4.2.3.

Precast concrete pipe shall comply with the requirements of Publication 280.

12.4.2.4 Precast Concrete Structures

The following shall replace A12.4.2.4.

Precast concrete arch, elliptical and box structures shall comply with the requirements of Publication 408.

12.4.2.9P Bolts for Structural Plate Structures

The construction of structural plate structures requires field assembly of the individual plates to form the metal shell around which the soil envelope is to be compacted.

C12.4.1.3

The following shall replace AC12.4.1.3.

In-situ soil along the sides of the structure need not be excavated (except for placement and compaction of the minimum width of structural backfill) and recompact unless the quality of the in situ-soil is not comparable to the proposed compacted side fill.

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The fasteners used for assembly shall be 3/4 in. diameter galvanized steel, stainless steel, or aluminum bolts. The underside of the bolt heads and nuts to be used for assembling structural plate structures shall be uniformly rounded to permit installation on either the crown or valley of the corrugations without stress variation. Washers shall not be required for the assembly of corrugated metal plate structures.

Bolts used for fastening aluminum structural plates shall meet the requirements of ASTM A 307 for steel bolts, and ASTM F 468 for aluminum bolts. Bolts used for fastening steel structural plates shall meet the requirements of ASTM A 449. The galvanizing used on bolts for either aluminum or steel structural plate shall meet the requirements of ASTM A 153. In severely corrosive environments, stainless steel bolts may be used to assemble corrugated aluminum structural plates. Stainless steel nuts and bolts shall meet the requirements of ASTM A 193M.

Hook and anchor bolts shall be used to provide a connection between the ends of a structural plate structure and the headwalls, where applicable. Hook or anchor bolts shall be placed in alternate bolt holes. Hook and anchor bolts shall not be used as fasteners on the end of a longitudinal seam. These bolts and their galvanized coatings shall meet the same criteria as the bolts used to assemble the structure. The nuts for hook and anchor bolts shall meet the requirements of ASTM A 563, Grade C, for steel fasteners and ASTM F 467 for aluminum fasteners.

12.5 LIMIT STATES AND RESISTANCE FACTORS**12.5.5 Resistance Factors**

The following shall replace Table A12.5.5-1.

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Table 12.5.5-1 – Resistance Factors for Buried Structures

STRUCTURE TYPE	RESISTANCE FACTOR
Metal Pipe, Arch and Pipe Arch Structures	
Helical pipe with lock seam or fully-welded seam: • Minimum wall area and buckling	1.0
Annular pipe with spot-welded, riveted or bolted seam: • minimum wall area and buckling • minimum seam strength	1.0 0.67
Structural plate pipe: • minimum wall area and buckling • minimum seam strength • bearing resistance of pipe arch foundations	1.0 0.67 refer to D10.5.5
Long-Span Structural Plate and Tunnel Liner Plate Structures	
• minimum wall area • minimum seam strength • bearing resistance of pipe arch foundations	0.67 0.67 Refer to D10.5.5
Structural Plate Box Structures	
• plastic moment strength • bearing resistance of pipe arch foundations	1.0 refer to D10.5.5
Reinforced Concrete Pipe	
Direct design method: • flexure • shear • radial tension	0.90 0.85 0.85
Reinforced Concrete Cast-in-Place Box Structures	
• flexure • shear	0.90 0.85
Reinforced Concrete Precast Box Structures	
• flexure • shear	1.00 0.90
Reinforce Concrete Precast Three-Sided Structures	
• flexure • shear	0.95 0.90
Thermoplastic Pipe	
PE and PVC pipe: • minimum wall area and buckling • flexure	1.00 1.00

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12.5.6 Flexibility Limits and Construction Stiffness**C12.5.6P**

The following shall supplement A12.5.6.

The flexibility factor, FF, in in./kip, of metal pipe shall be calculated as:

$$FF = \frac{D_e^2}{EI} \quad (12.5.6-1P)$$

where:

I = moment of inertia (in.⁴/in.)

E = long-term modulus of elasticity of pipe material (ksi)

D_e = effective diameter of pipe (in.)

The construction stiffness factor, C_s, in kip/in., for steel tunnel liner plate shall be calculated as:

$$C_s = \frac{EI}{D_e^2} \quad (12.5.6-2P)$$

where:

I = moment of inertia (in.⁴/in.)

E = long-term modulus of elasticity of the steel tunnel liner plate (ksi)

D_e = effective diameter of pipe (in.)

12.5.6.2 Spiral Rib Metal Pipe and Pipe Arches

The following shall replace Table A12.5.6.2-1.

Table 12.5.6.2-1 - Flexibility Factor Limits

MATERIAL	CONDITION	CORRUGATION SIZE (in.)	FLEXIBILITY FACTOR (in/kip)
Steel	Embankment	0.75 x 0.75 x 7.5	217 I ^{1/3}
		0.75 x 1.0 x 11.5	140 I ^{1/3}
Aluminum	Embankment	0.75 x 0.75 x 7.5	340 I ^{1/3}
		0.75 x 1.0 x 11.5	175 I ^{1/3}

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12.6 GENERAL DESIGN FEATURES**12.6.2 Service Limit State****12.6.2.1 Tolerable Movement**

The following shall supplement A12.6.2.1.

For elliptical and round corrugated metal pipe, the crown deflection shall not exceed δ_c , given by:

$$\delta_c = \mu_m \frac{D_h^2}{d} \quad (12.6.2.1-1P)$$

where:

δ_c = crown deflection (in.)

μ_m = shape factor (dim)

D_h = horizontal span of pipe (in.)

d = depth of corrugation (in.)

The value of the shape factor is obtained from Figure 12.6.2.1-1P.

For long-span structures, the following deformation criteria shall apply:

- For horizontal ellipse shapes having a ratio of top to side radii of three or less, the span and rise shall not deviate from the specified dimensions by more than 2%.
- For arch shapes having a ratio of top to side radii of three or more, the rise shall not deviate from the specified dimensions by more than 1% of the span.
- For all other long-span structures, the span and rise shall not deviate from the specified dimensions by more than 2%, nor more than 5 in., whichever is less.

For structural plate box structures, the rise shall not deviate from the specified dimensions by more than 1% of the span.

For other corrugated metal pipe shapes and thermoplastic pipes, the crown deflection shall not exceed 5% of the rise, unless otherwise approved.

C12.6.2.1P

During the initial stages of backfilling, when the pipe is not fully contained by the soil, the pipe wall undergoes high deformations. These, if allowed to exceed certain limits, will include permanent set in the wall. The provision in this clause is aimed at restricting such excessive deformations.

The limits of vertical crown deflections for round and elliptical pipes were derived by an analysis in which the pipe walls, with a 6 in. x 2 in. corrugation profile, were subjected to uniform lateral pressures. Limits of the crown deflections, which could be characterized by the span-to-rise ratios of the pipe, were established so that the stresses anywhere in the cross-section remained within 90% of the yield stress of the steel. Values of the non-dimensional parameter μ_m , given in Figure 12.6.2.1-1P, were calculated from the results of the above analysis. For shapes other than round and elliptical, the limit of the vertical deflection is based mostly on practical considerations, rather than on analysis.

The crown deflection limits, specified in the clause, are meant for construction rather than design control. If the deformations begin to exceed the specified limits, measures should be taken to contain them. With the present state-of-the-art, it is difficult to "design" a pipe wall so that it can comply with the provision of this article.

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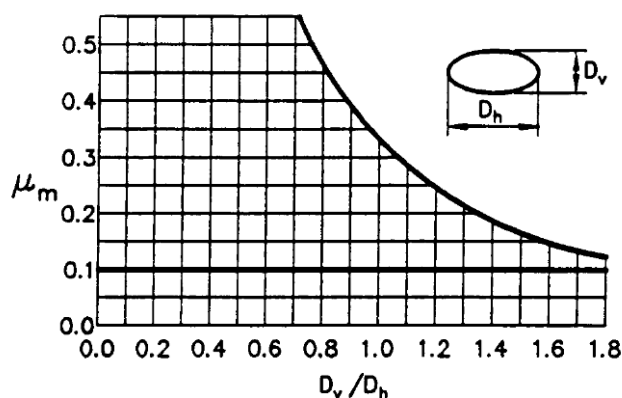


Figure 12.6.2.1-1P - Values of μ_m for Elliptical and Round Corrugated Metal Pipe (Ontario Ministry of Transportation, 1992)

12.6.2.2 Settlement

12.6.2.2.1 General

The following shall supplement A12.6.2.2.1.

- differential settlement between the pipe and endwall

When the foundations for soil-steel structures have markedly non-uniform settlement properties within the extent of the pipe and its structural backfill, appropriate measures, including the removal of unsuitable materials, shall be taken to avoid their detrimental effects on the structure.

12.6.2.2.2 Longitudinal Differential Settlement

The following shall replace the second sentence in the first paragraph of A12.6.2.2.2.

Pipes and culverts subjected to longitudinal differential settlements shall be fitted with positive joints to resist disjoining forces.

12.6.2.2.4 Footing Settlement

The following shall supplement A12.6.2.2.4.

Metal box culverts shall not be used where foundation soil or loading conditions are likely to lead to differential settlement between footings.

C12.6.2.2.1P

Foundations with reasonably uniform settlement properties are necessary to avoid situations where the invert may be founded partially on compressible materials and partially on incompressible materials. Lack of uniformity along the invert bedding could induce undesirable stress concentrations in the pipe wall above the incompressible areas. A condition in which the foundation under the pipe is less compressible than that of the adjacent areas should be avoided. In such conditions, columns of soil adjacent to the pipe settle more than the column of soil above the pipe, thus inducing negative arching which, in turn, increases the thrust. Foundations with fairly uniform settlement properties can be provided by adjusting the bedding thickness, replacing compressible materials, shattering bedrock, and other similar treatments.

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12.6.2.2.5 *Unbalanced Loading*

The following shall supplement A12.6.2.2.5.

Due to the complexity of determining the actual load distribution on a structure subjected to unbalanced loading, the problem can be modeled using numerical methods or approximated as an edge beam. If the edge beam method is used, the provisions of A4.6.2.1.4b for longitudinal edges shall be applied and checked along the skewed section (see Figure 12.6.2.2.5-1P) and the normal main reinforcement shall be provided as shown in Figure 12.6.2.2.5-1P.

Refer to Appendix H "Pennsylvania Installation Direct Design (PAIDD) for Concrete Pipes" for the design of concrete pipes subjected to unbalanced loading.

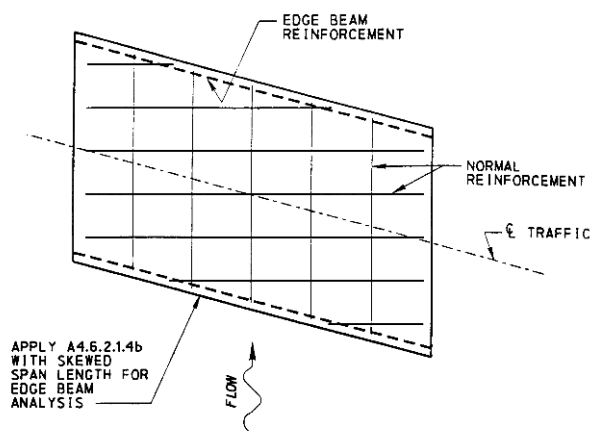


Figure 12.6.2.2.5-1P - Reinforcement Requirement for Box Culvert with Skew Ends

12.6.2.2.6P *Structure on Unyielding Foundation*

Buried structures shall not be placed directly on an unyielding foundation. Unless determined by a special analysis (e.g., finite element analysis), a minimum 2 ft. thickness of soil bedding shall be placed between the foundation and the bottom of the structure.

12.6.2.2.7P *Endwall Settlement*

The design of headwalls shall provide sufficient restraint to maintain the shape of the structure under the design loads. The footings of headwalls and wingwalls shall be proportioned so that the calculated settlement of the corrugated metal shell shall be the same as the calculated settlement of the headwalls and endwalls.

12.6.2.3 *Uplift*

The following shall supplement A12.6.2.3.

To satisfy this condition, the dead load on the crown of

COMMENTARY

C12.6.2.2.5

The following shall supplement AC12.6.2.2.5.

Skewed culverts are subjected to horizontally unbalanced soil pressures in the sloped embankment range. This unbalanced distributed force has to be carried by the culvert and by the headwalls (if headwalls are provided). Consider shear if the headwalls are connected to the culvert. In the case of a skewed culvert with shear connected headwall, the culvert is restrained by the headwall from deflecting laterally to carry the unbalanced horizontal load by bending. A support system (as shown in Figure AC12.6.2.2.5-1) may develop which carries the unbalanced distributed load, $E(x)$, by a horizontal shear component, F , acting between headwall and culvert, and shear forces, S_1 and S_2 , acting along the culvert bearing lines.

C12.6.2.2.6P

The effect of constructing a buried structure directly on an unyielding foundation is an amplification of the vertical earth load on the culvert.

C12.6.2.3

Delete AC12.6.2.3.

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the structure shall exceed the buoyancy of the culvert, drainage shall be provided to maintain the groundwater at a level below the culvert, or the structure shall be anchored to resist uplift forces. Whenever practical, the drainage option shall be used.

12.6.5 Scour

The following shall supplement A12.6.5.

Cutoff walls or scour curtains shall extend to a minimum depth of 3'-6" below all pipe inlets and outlets and arch structure footings placed over erodible deposits in accordance with Standard Drawing BD-631M.

12.6.6 Soil Envelope**12.6.6.1 Trench Installations****C12.6.6.1**

The following shall replace the first paragraph of AC12.6.6.1.

Refer to Standard Drawing RC-30M for guidance regarding minimum trench width.

12.6.6.2 Embankment Installations**C12.6.6.2**

The following shall supplement A12.6.6.2.

Refer to Standard Drawing RC-30M for guidance regarding minimum soil envelope widths.

Delete AC12.6.6.2.

12.6.6.3 Minimum Soil Cover

The following shall supplement A12.6.6.3.

The minimum soil cover for thermoplastic pipe shall be the larger of 2 ft., the pipe ID, or the greater of minimum cover as defined by Standard Drawing RC-30M or the tables in Chapter 10 of Design Manual, Part 2.

For construction requirements for thermoplastic pipes, refer to Standard Drawing RC-30M (based on ASTM D 2321). Pipe is not to be placed in the pavement structure without Chief Bridge Engineer approval.

12.6.6.4P Maximum Soil Cover

See Standard Drawing BD-635M and Design Manual, Part 2, for height of cover limits for metal culverts and thermoplastic pipes.

12.6.7 Minimum Spacing Between Multiple Lines of Pipe**C12.6.7**

The following shall supplement A12.6.7.

Allow 2 ft. distance for minimum spacing requirements between multiple lines of pipe. Refer to Standard Drawing BD-636M and Appendix H, "Pennsylvania Installation Direct Design (PAIDD) for Concrete Pipes" for minimum spacing requirements between multiple lines of concrete pipe.

Delete AC12.6.7.

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12.6.8 End Treatment**12.6.8.1 General**

The following shall supplement A12.6.8.1.

Refer to Standard Drawings BD-631M, BD-632M, BD-633M, BD-634M, RC-30M, RC-31M and RC-33M for end treatment requirements for buried structures.

A culvert with a skew of 75 degrees and a two horizontal to one vertical (2H:1V) bevel shall not be exceeded without end reinforcement. Square ends may be designed with side plates beveled up to a maximum 2H:1V slope without reinforcement. Skew ends up to 75 degrees with no bevel are permissible. Skew ends on spans over 20 ft. shall be protected with a reinforced concrete headwall or structural steel collar. When partial headwalls are skewed, the offset portion of the metal structure shall be supported by the headwall. The maximum skew shall be limited to 55 degrees.

For hydraulic structures, additional reinforcement of the end is recommended to secure the metal edges at the inlet and outlet against hydraulic forces. Reinforced concrete, structural steel collars, tension tiebacks or anchors in soil, partial headwalls, or cutoff walls below the invert elevation are methods which may be considered. See Figure 12.6.8.1-1P for typical slope wall for metal culverts.

Beveled ends shall be anchor-bolted to headwalls or slope walls at approximately 18 in. intervals. Square-end structures may be anchored to a headwall with bolts in alternate standard circumferential holes.

Reinforced concrete headwalls shall be provided for concrete pipes greater than 8 ft. in diameter.

End treatments for thermoplastic pipe are presented in Standard Drawing RC-31M.

The type of end treatment for culverts shall be approved by the District Bridge Engineer.

C12.6.8.1

The following shall supplement AC12.6.8.1.

Square ends are the most common and generally most economical ends, and are used when the structure is installed normal to a roadway. Unless required by the velocity of water through the conduit, no further inlet or outlet treatment is needed.

Because the structural capacity of a long-span structure is derived from the interaction of the corrugated metal shell and the surrounding soil, it is very important to evaluate the loading effects at the free ends. Whenever possible, the free ends should be located at a distance equal to the height of fill from the edge of a trafficable shoulder.

End structures for culvert inlets serve to improve the hydraulic capacity of the structure and reduce the potential for scour and piping. Inlets should provide a smooth transition from the channel to the culvert to accomplish the following:

- Minimize hydraulic head losses due to turbulence
- Prevent or minimize the contraction of the water channel entering the culvert barrel

Potential methods for achieving a smooth transition and inlet protection include the following:

- Riprap extending to the level of the maximum flow elevation
- Recessing the invert slightly below the streambed elevation to prevent undercut
- Construction of a headwall, or headwall and wingwall

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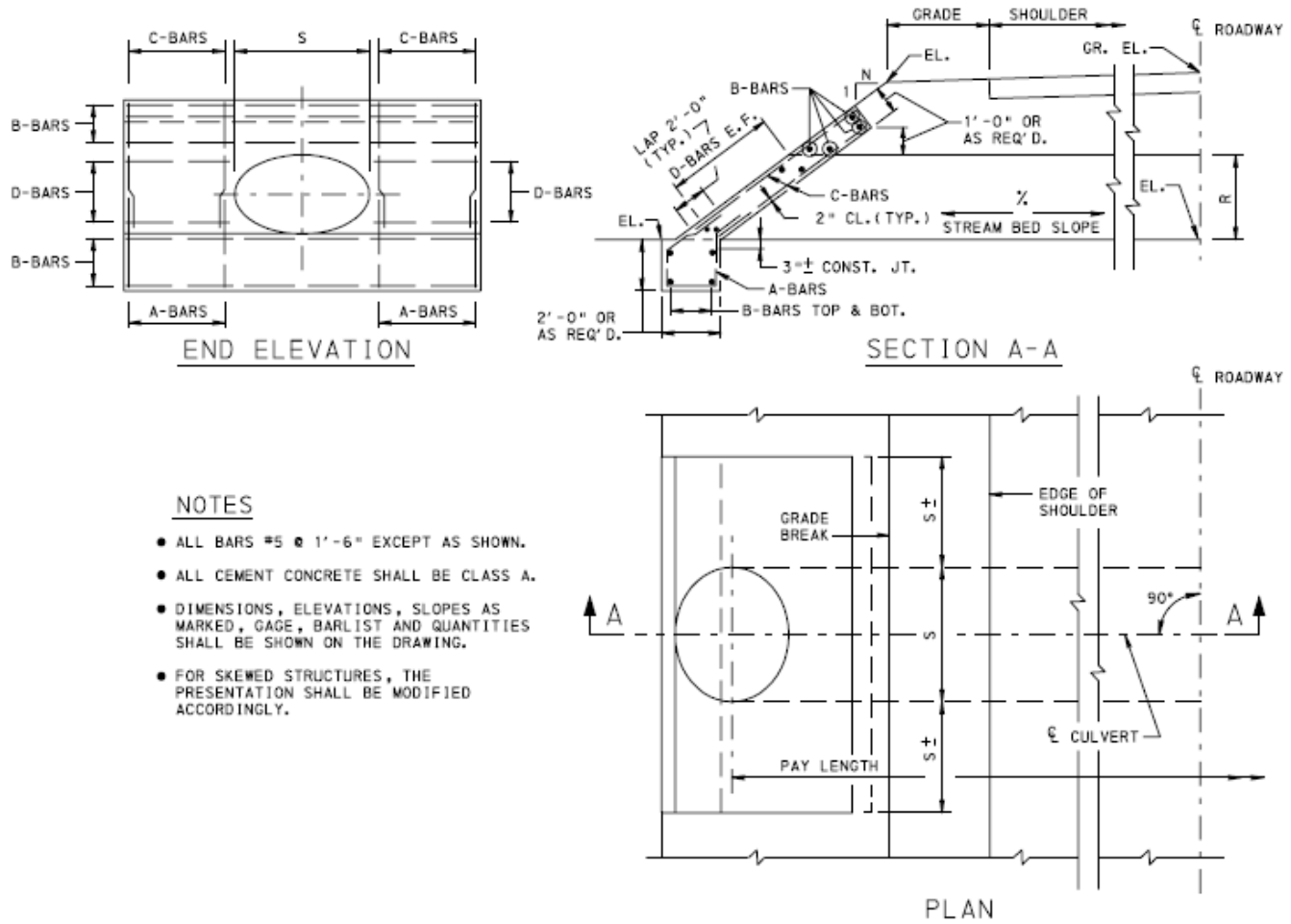


Figure 12.6.8.1-1P - Slope Wall for Metal Culverts

12.6.8.2 Flexible Culverts Constructed on Skew

The following shall supplement A12.6.8.2.

Disregarding the effect of lateral unbalanced forces during headwall design may lead to failure of the headwall and adjacent culvert sections.

Unless otherwise directed, the angle of end walls shall be in accordance with Table 12.6.8.2-1P. The angle of end walls shall be shown on the preliminary plans submitted for type, size and location approval.

When the end wall is not parallel to the edge of shoulder, the embankment slope shall be warped to cover the exposed portion of the structure. Show on the plans when warping of embankment slope is necessary, similar to the grading shown in Figure AC12.6.8.2-1.

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Table 12.6.8.2-1P – End Wall Angle

Fill Height at Edge Shoulders	Skew Angle (deg)	Angle of End Wall (deg)
≤ 4 ft.	Any	Parallel to edge of shoulders
4 to 8 ft.	90 – 75 75 – 60 less than 60	90* 75* 60
>8 ft.	90 – 60 less than 60	90* 60

*If shallow fills do not permit warping of the embankment slopes to eliminate exposure of the culvert, end walls shall be constructed parallel to edge of shoulders.

The structural design provisions, herein, are intended for square-ended structures with uniform backfill and surface loading. Deviation from these conditions shall be evaluated. Table 12.6.8.2-2P provides guidance for a variety of skew and bevel conditions.

Table 12.6.8.2-2P - Requirements for Cut Ends without Headwalls (Kaiser Aluminum, 1985)

REQUIREMENTS FOR CUT ENDS WITHOUT HEAD WALLS
(Kaiser Aluminum, 1985)

No.	Cut End	Typical View of Installation		Requirements
		Plan	Section XX	
1	square end with roadway parallel to transverse direction			
2	square end with roadway skew to transverse direction			θ shall be less than 40° . For $\theta > 20^\circ$, the earth pressure imbalance shall be accommodated either by structural reinforcement of conduit wall or by contour grading of the embankment slope.
3	skew end			As for No. 2.
4	square bevel with roadway parallel to transverse direction			b shall not be less than (rise/8). The ends shall be treated as a retaining structure and shall be designed accordingly. b = bottom step
5	square bevel with roadway skew to transverse direction			As for Nos. 3 and 4.
6	skew bevel			Requires approval.

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12.6.8.3P Precast and Cast-in-Place Box Culverts

The ends of box sections shall be normal to the walls and centerline of the box sections. When a beveled end is specified at the culvert end, an edge beam shall be required.

12.6.8.4P Fish Passage Through Culverts

Fish passage shall be provided in culverts in fishable streams having continuous flow.

See Design Manual, Part 2, Chapter 10, for guidance regarding the design of low flow fish passage systems through culverts.

12.6.9 Corrosive and Abrasive Conditions

The following shall supplement A12.6.9.

Flexible corrugated metal structures shall be designed to provide the structural capacity required by the appropriate design loads throughout the design life of the structure. The corrugated metal plates shall include either a sacrificial thickness equal to the expected metal loss (due to corrosion and abrasion) or a protective coating which is capable of sustaining the metal structure.

Refer to D12.6.9.4P for acceptable protective coatings.

12.6.9.1P Corrosion

Corrosion of corrugated metal or reinforcing steel in concrete drainage structures is a major concern in the selection of the materials to be used for a buried structure. Indicators of corrosion potential include soil and water pH, resistivity, oxidation-reduction potential, chemical composition of backfill soils, precipitation, and flow velocity. As a minimum, the following information shall be developed for the culvert design:

- (a) pH of soil, surface water and groundwater
- (b) resistivity of soil, surface water and groundwater
- (c) sulfate content of the stream flow or runoff

See Table 12.6.9.1P-1 for typical resistivity values for soil and water. For general guidelines regarding soil corrosion, see Table 12.6.9.1P-2. For cases in which abrasion is not a problem, cathodic protection can be provided to reduce the corrosion potential of metal.

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Table 12.6.9.1P-1 - Typical Resistivity Values

Soil		Water	
Classification	ohm-cm	Source	ohm-cm
Clay	750-2000	Seawater	25
Loam (silts)	3000-10 000	Brackish	2000
Gravel	10 000-30 000	Drinking Water	4000+
Sand	30 000-50 000	Surface Water	5000+
Rock	50 000-Infinity*	Distilled Water	Infinity*

*Theoretical

Source: Adapted from AASHTO (1993)

Table 12.6.9.1P-2 - Corrosiveness of Soils

Soil Type	Description of Soil	Aeration	Drainage	Color	Water Table
I Lightly Corrosive	Sands or sandy loams Light textured silt loams Porous loams or clay loams thoroughly oxidized to great depths	Good	Good	Uniform Color	Very Low
II Moderately Corrosive	Sandy Loams Silt Loams Clay Loams	Fair	Fair	Slight Mottling	Low
III Badly Corrosive	Clay Loams Clays	Poor	Poor	Heavy Texture Moderate Mottling	2 to 3 ft. below surface
IV Unusually Corrosive	Muck Peat Tidal Marsh Clays and Organic Soils	Very Poor	Very Poor	Bluish-Gray Mottling	At surface or extreme impermeability

Source: (Hurd, 1984)

12.6.9.2P Abrasion

Waters with suspended bed loads (i.e., sand-, gravel- and cobble-size particles) flowing at high velocities (i.e., greater than about 10 ft/sec) and creating appreciable turbulence can cause severe localized deterioration resulting from the combined action of mechanical abrasion and corrosion. Known as corrosion-abrasion, corrosion-erosion, or erosion-corrosion, this includes both impingement attack and cavitation. These effects are very difficult to separate from corrosion in metal culverts as the two often work in conjunction with each other.

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12.6.9.3P Materials

Those materials subject to deterioration by corrosion and abrasion which are used for buried structures include steel, aluminum and concrete. Other culvert materials which are not considered in A12.6.9 and D12.6.9 here include vitrified clay, stainless steel, cast iron and thermoplastic.

12.6.9.3.1P Steel

Recommended conditions for the installations of corrugated galvanized or aluminized steel buried structures include:

- (a) Soil and water pH within the range of 5.5 to 8.5
- (b) Resistivity greater than 6000 ohm-cm

12.6.9.3.2P Aluminum

Aluminum is suitable for use in neutral to mildly acidic environments and performs well in organic acid environments. Recommended conditions for the installation of corrugated aluminum drainage structures include:

- (a) Soil and water pH within the range of 4 to 8.5
- (b) Resistivity greater than 500 ohm-cm

12.6.9.3.3P Concrete

Concrete culverts, because of their rigidity, are susceptible to foundation movements which can expose steel reinforcement to corrosion. Concrete surfaces exposed to flow and soil are susceptible to sulfate attack and abrasion. The use of high compressive strength concrete and durable, hard aggregate increases abrasion resistance. Concrete is resistant to many chemicals, but is subject to acid attack, especially sulfuric and sulfurous acids from acid mine drainage. Acceptable effluent limitations for concrete culvert include:

- (a) pH greater than 4.0
- (b) Sulfate content in solutions within the range of 100 to 1000 ppm

Rubber gaskets for circular pipe connections may be used in the following situations:

- (a) Whenever foundation conditions are conducive to differential movement between sections
- (b) If the culvert is placed under more than 15 ft. of fill

C12.6.9.3.1P

For installation in mildly acidic environments, uncoated galvanized steel is less suitable than aluminum.

C12.6.9.3.2P

On aluminum surfaces, as with other metals, an oxide film develops which increases the metal's resistance to corrosion. Abrasion will remove this film and allow an acceleration of corrosion which may or may not reform the oxide film.

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- (c) If the culvert is constructed on an embankment
- (d) If joint failure could be difficult or expensive to correct

Whenever box culvert or vertical or horizontal elliptical culvert sections are specified, foundation conditions shall be stabilized to reduce the potential of differential settlement between culvert sections. Vitrified clay liner plates may also be specified in accordance with PP3.4.5 for corrosion protection.

12.6.9.4P Protective Coatings

Because environmental conditions at drainage structure locations do not always meet the recommended criteria for aluminum, concrete, or steel buried structures, coatings can be added to protect the structure from corrosion and abrasion.

Acceptable protective coatings include the following:

- Shop- or field-applied coatings applied in accordance with AASHTO M 190, M 224 and M 245M for metal and concrete pipe products.
- Sacrificial metal plate thickness for corrugated metal structures.
- Class A cement concrete paving in the bottom of pipes or pipe arches in accordance with Publication 408, Section 603.

Refer to Design Manual, Part 2, and Standard Drawing BD-635M for guidance regarding specific applications and limitations.

No service life credit shall be given for unreinforced concrete lining of corrugated metal culverts.

12.6.9.5P Performance Estimates

Mathematical formulas and graphical charts have been developed, on the basis of field observations, to estimate the metal loss rate of galvanized corrugated steel culverts and structural steel plate culverts, as presented in Figures 12.6.9.5P-1 and 12.6.9.5P-2. The minimum metal loss rates shall be 2 mil/year for galvanized metal pipes and 1 mil/year for aluminized and aluminum metal pipe. The minimum predicted metal loss rate specified in Figures 12.6.9.5P-1 and 12.6.9.5P-2 shall be added to the required structural wall thickness to provide the required service life of the corrugated steel culverts.

When pH and resistivity data is available, Figure 12.6.9.5P-3 can be used to estimate the years to perforation

C12.6.9.4P

The Department does not generally use protective coatings on concrete pipe.

For concrete culverts with less than 2 ft. of cover, protective coating may be needed and should be applied to the entire outside face of the top and sides of the culvert

Concrete lining of corrugated metal pipe provided to improve hydraulic efficiency has been observed to crack or break up after installation. This thin lining is not expected to extend the service life of the culvert.

C12.6.9.5P

Extensive field studies by New York State Department of Transportation and other states show that the minimum metal loss rates are reasonable.

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and to indicate the inspection and maintenance interval of corrugated steel culverts.

Similarly, mathematical formulas and graphical charts have been developed for concrete culverts to estimate the culvert service life as presented in Figure 12.6.9.5P-4. A correction factor is to be applied to the culvert service life because of the effect of sediment depth on the life of the concrete culverts. This correction factor is presented in Figure 12.6.9.5P-5.

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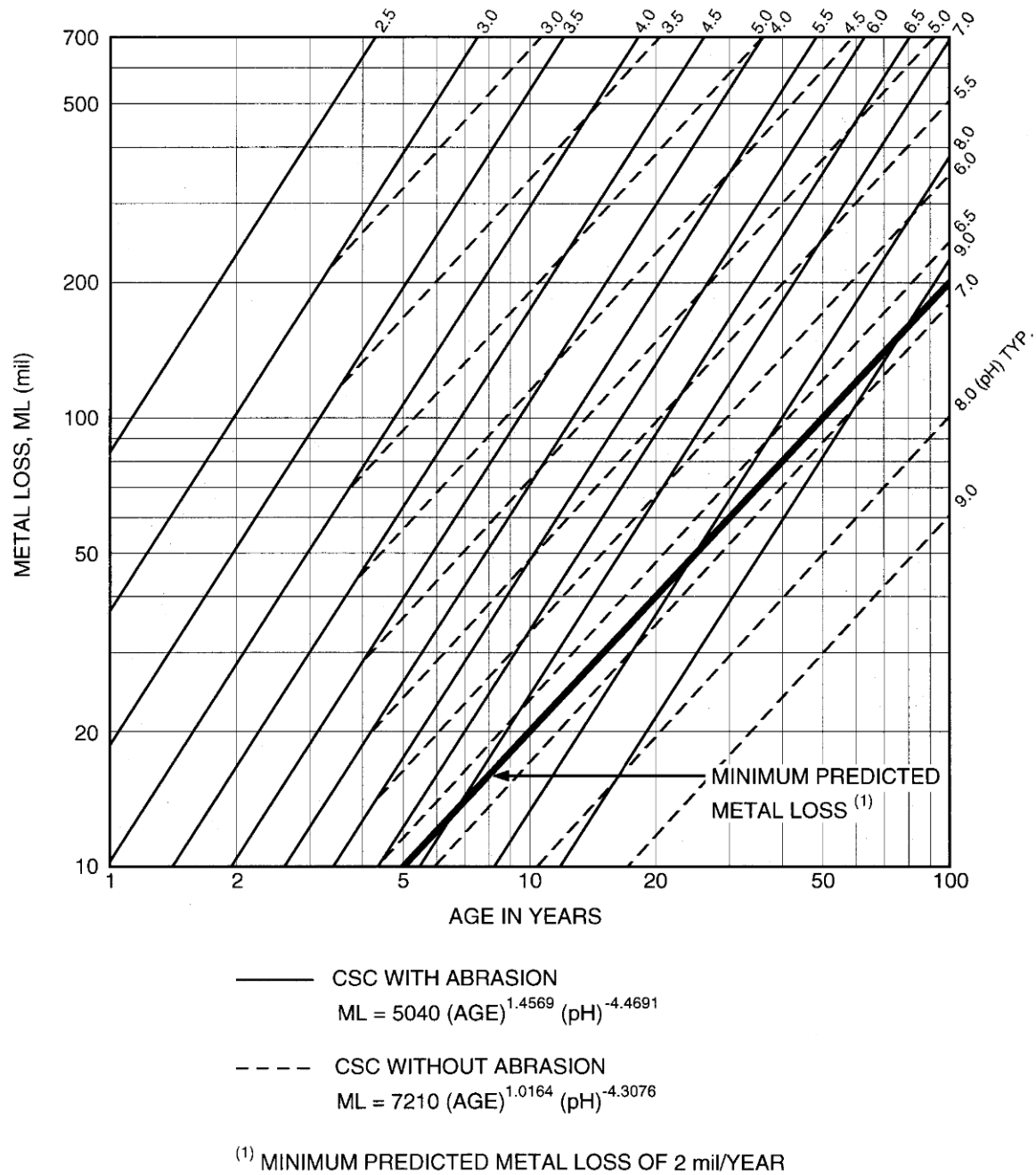


Figure 12.6.9.5P-1 – Predicted Metal Loss Rate for Corrugated Steel Culvert (CSC), AASHTO M 36/M 36M (modified after Hurd, 1984)

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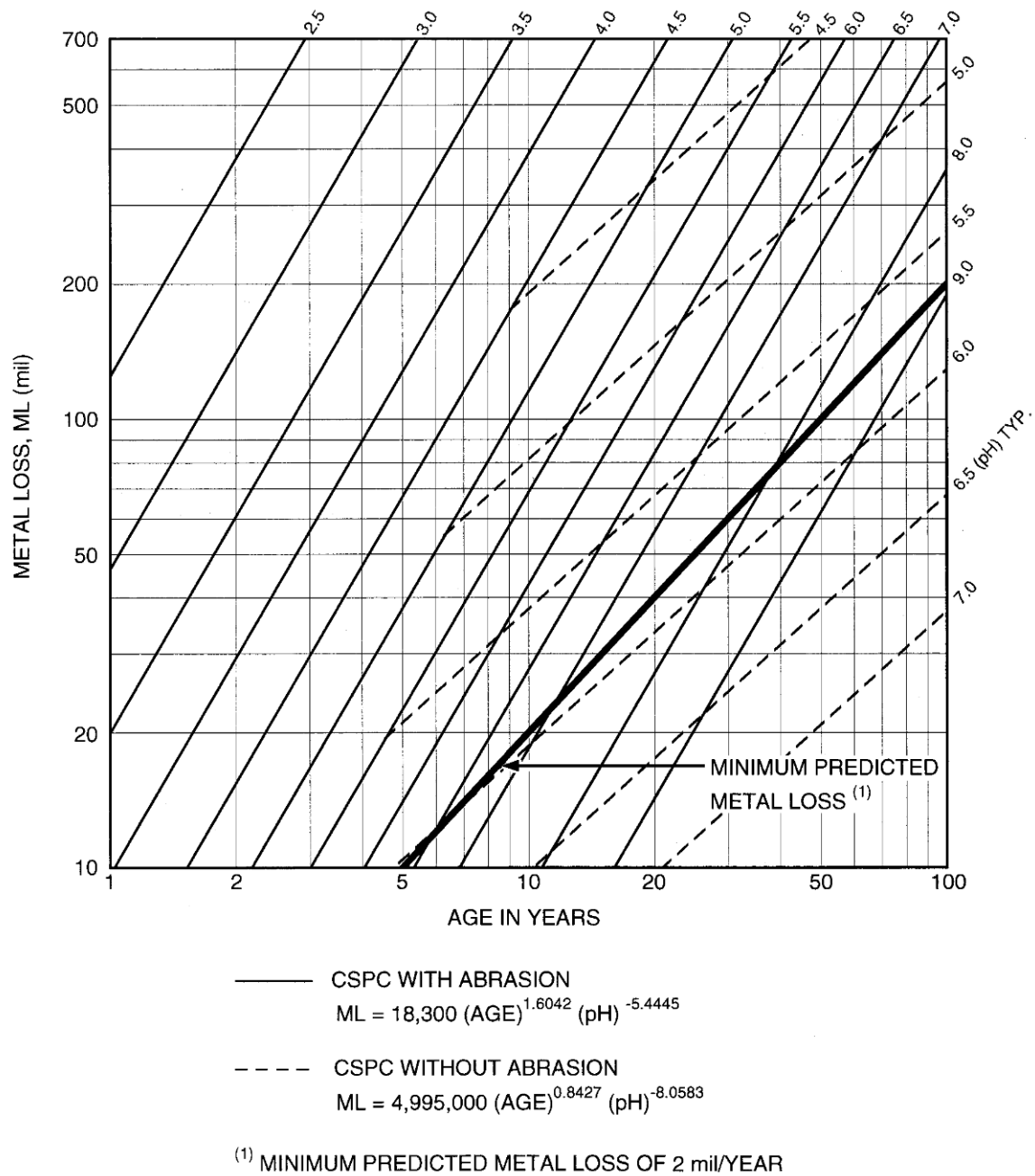


Figure 12.6.9.5P-2 – Predicted Metal Loss Rate for Corrugated Steel Plate Culvert (CSPC), AASHTO M 167/M 167M (modified after Hurd, 1984)

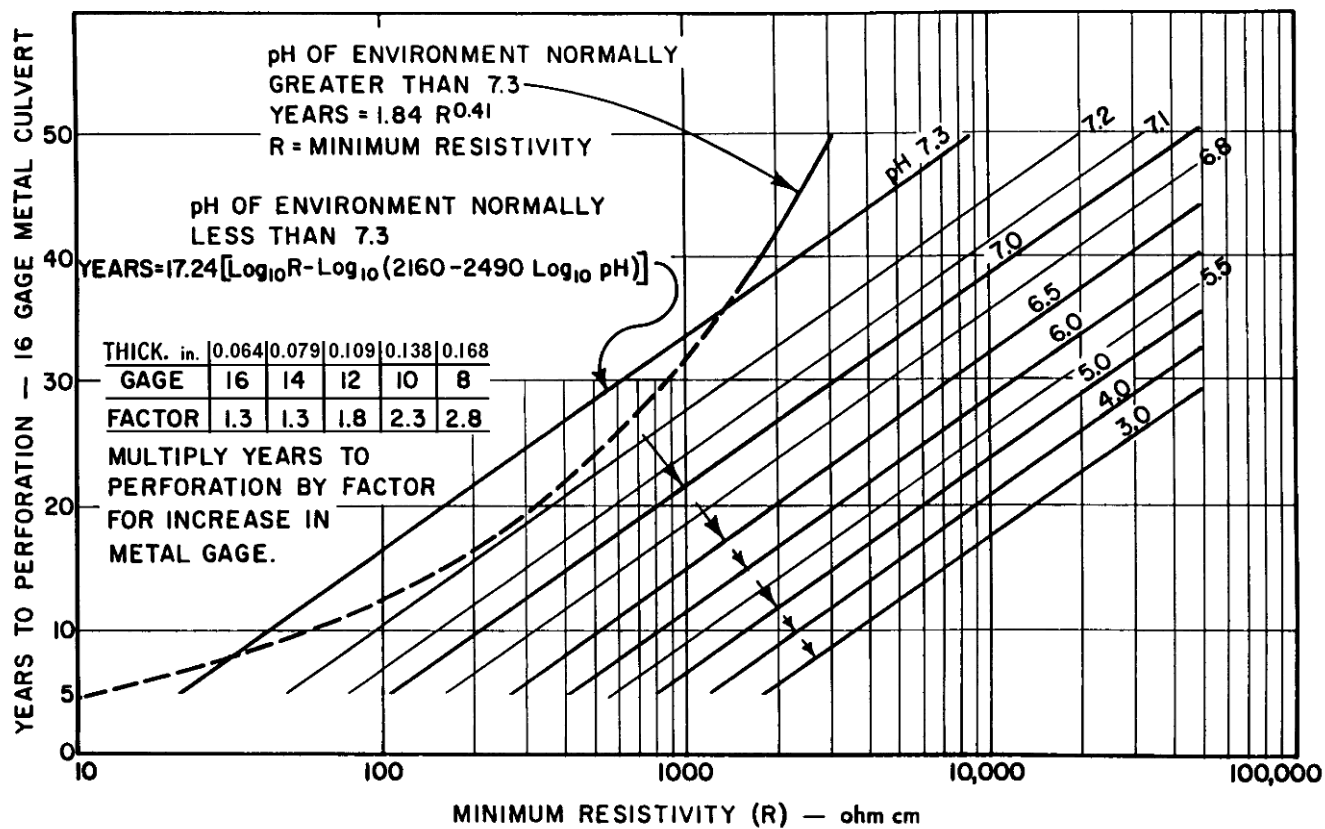
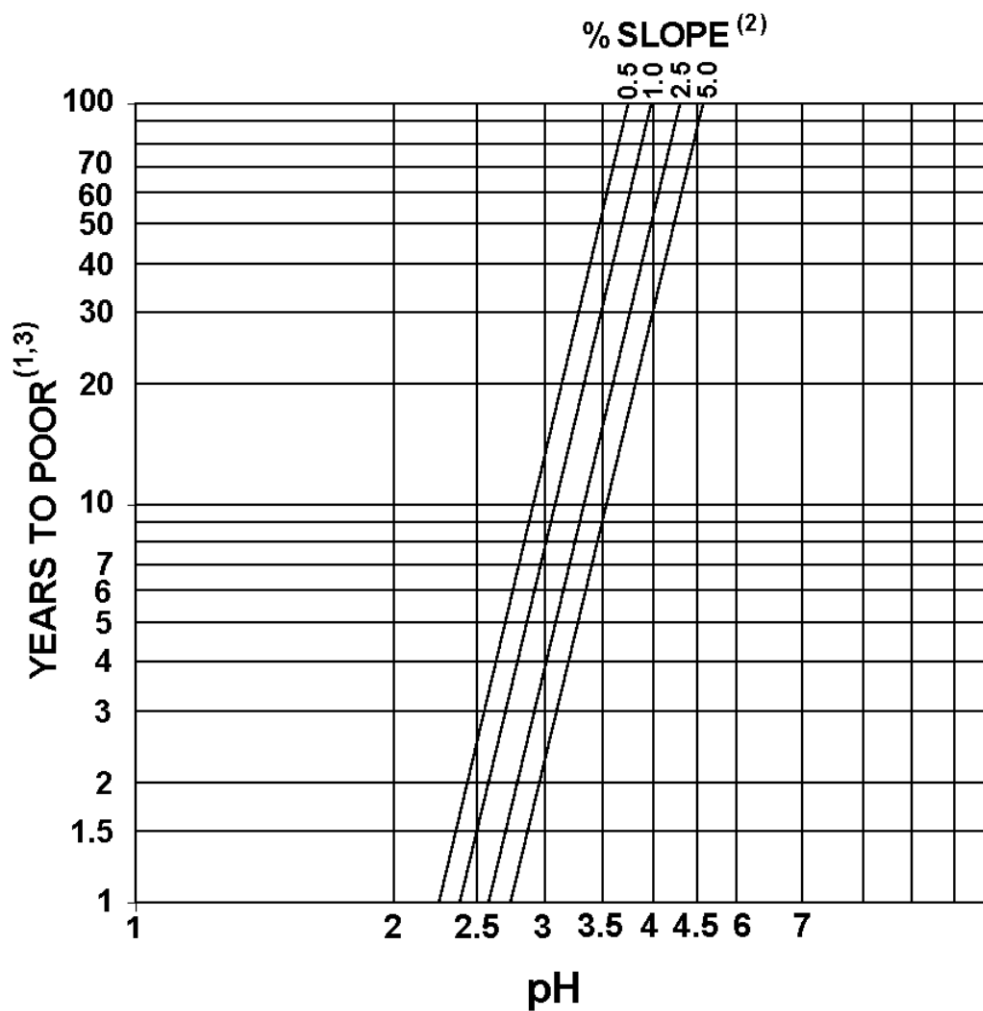


Figure 12.6.9.5P-3 - Estimated Maintenance Interval of Corrugated Steel Pipe (modified after TRB, 1978)



(1) Significant loss of mortar and aggregate; complete loss of invert; concrete in softened condition

(2) Relationships assume no accumulation of sediment

$$(3) \text{ Years to Poor} = \frac{(0.3509(\text{pH})^{1.205})^{7.457} \left(1 + \frac{\text{sediment depth}}{\text{rise}}\right)^{8.630}}{(\text{slope})^{0.767}}$$

Figure 12.6.9.5P-4 – Predicted Concrete Culvert Life (Meacham, et al, 1982)

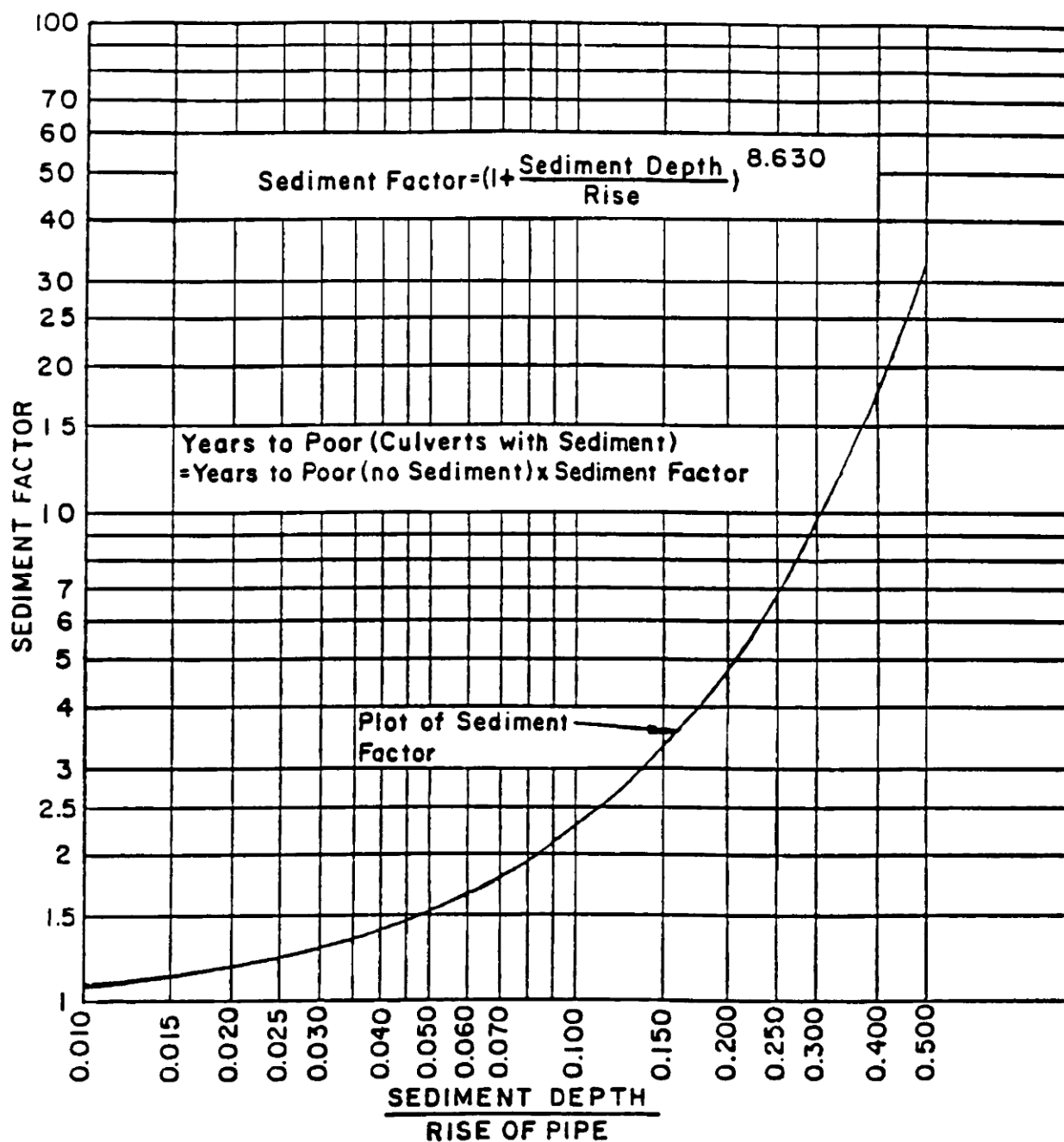


Figure 12.6.9.5P-5 - Effect of sediment depth on life of concrete culverts (Meacham, et al, 1982)

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An abrasion resistance performance rating schedule was developed with respect to culvert diameter, mean culvert water velocity, rock size, and culvert slope to estimate aluminum drainage structure service life (see Figure 12.6.9.5P-6 and Table 12.6.9.5P-1). The mean culvert water velocity, V_w , for this analysis is presented in Table 12.6.9.5P-2.

A minimum allowable metal loss rate of 1 mil/year per year shall be added to the required structural wall thickness to provide the required service life of aluminum and aluminized culverts.

Table 12.6.9.5P-1 - Abrasion Performance Rating Schedule
(Koepp and Ryan, 1986)

Performance Zone Ratings	Effect on Surface of Crown of Corrugation, Invert Only*
A	No surface effect - No reduction in service life due to bed load abrasion. Projected Abrasion Service Life, 100 years or more.
B	Non-erosive - Some slight roughening of the metal surface, but no metal removal by erosion action. No reduction in normal service life of aluminum culvert. Projected Abrasion Service Life, 75 years or more.
C	Erosion - Surface roughening and slight progressive removal of metal from culvert. Some gouging may be noted if rocks tend to be large. Projected Abrasion Service Life, 50 years or more.
D	Abrasion - Surface roughening and slow removal of metal from culvert. Definite reduction in pipe life due to abrasion. Gouging of surface may be expected. Projected Abrasion Service Life, 25 to 50 years.
E	Abusive - Surface roughening and rapid removal of metal from culvert. Definite reduction in pipe life due to abrasion. Projected Abrasion Service Life, 25 years or less.

*Abrasion affects only this portion of the surface. The remainder of the culvert is usually unaffected.

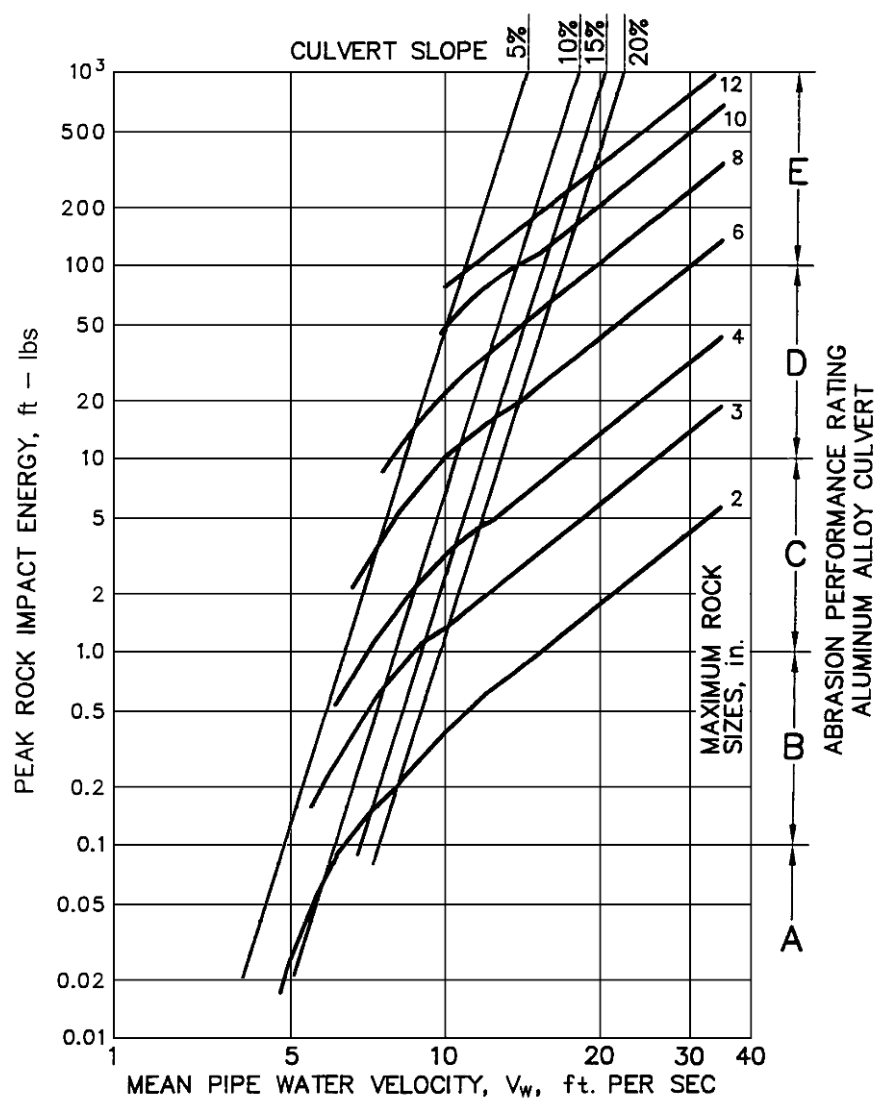


Figure 12.6.9.5P-6 – Abrasion Performance Rating for Aluminum Culvert (modified after Koepf and Ryan, 1986)

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Table 12.6.9.5P-2 – Assumed Mean Culvert Water Velocity for Abrasion Performance Rating for Circular Aluminum Culvert. Manning n-value is 0.24 (modified after Koepf and Ryan, 1986)

Dia. (in.)	Q (cfs)	Culvert Slope in %					
		5	10	15	20	25	30
18	2.0	5.5	6.6	7.6	8.3	9.1	10.0
24	4.4	6.2	8.0	9.3	10.6	11.5	12.0
30	8.0	7.2	9.4	10.7	12.0	12.8	14.0
36	13.0	8.0	10.3	12.0	13.5	14.2	15.1
42	20.0	10.0	12.0	14.0	15.1	16.5	17.8
48	29.0	10.1	13.5	15.0	16.5	18.0	19.5
60	52.0	11.7	15.5	17.5	20.0	21.7	23.0
72	83.0	13.2	17.0	20.0	21.3	24.0	25.5
84	130	14.8	19.0	21.5	24.0	26.0	27.7
96	186	16.0	21.0	24.0	26.8	29.0	31.0
		Table gives mean culvert velocity in fps					

12.6.10P Life-Cycle Cost

The life-cycle cost of a structure is a method used to compare various alternative buried structures to determine the comparative cost of each material and the method of installation. Life-cycle cost comparison methods involve the principles of engineering economics to determine the "present worth" of a structure, with consideration given to initial cost, maintenance costs, replacement costs, and the residual value of the structure at the end of the project design life. This method of analysis utilizes the parameters service life, interest rate and inflation rate to determine the most economical structure.

12.6.10.1P Service Life Estimation

Service life is defined as the number of years of good performance with minor maintenance. A structure may have reached its service life, but still be years from ultimate failure. Determination of the service life of a structure is dependent on durability, which is a function of the intended use of the structure; the relationship of the structure to its environment; the geometric constraints of the installation; and the site and environmental conditions, as discussed in D12.6.9.

The estimation of service life should include allowances for progressive changes in the culvert materials due to corrosion, abrasion and age, and in methods of installation, inspection and maintenance. Use of corrugated steel or

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aluminum culverts shall be restricted to secondary roads. For secondary road facilities, the minimum service life shall be 50 years. For primary highway facilities or for secondary road facilities where replacement access is limited (e.g., below high embankment fills), the minimum service life shall be 100 years. Pipe selection criteria are provided in Design Manual, Part 2.

Methods used to estimate the service life include the following:

- (a) Field performance surveys
- (b) Field prototype tests
- (c) Laboratory test methods
- (d) Analytical methods

12.6.10.1.1P Corrugated Metal

The design procedure for determining the gage or thickness of corrugated steel, steel structural plate and aluminum buried structures is based on the estimated metal loss field performance studies, pH, resistivity, and service life. Determination of the structural and hydraulic requirements for structure installation will result in a metal thickness necessary to support local loading conditions. The durability design requirements will utilize the complete and uniform predicted metal loss determined from Figures D12.6.9.5P-1 and D12.6.9.5P-2 for corrugated steel culverts and structural steel plate culverts, respectively, which correlates the required culvert age with the pH and abrasive nature of the water it will convey. This corresponds to the metal loss along the length of the culvert invert or flow line. The minimum predicted metal loss will be 2 mil/year as shown in Figures D12.6.9.5P-1 and D12.6.9.5P-2. The minimum predicted metal loss for aluminum and aluminized culverts is 1 mil/year. Once the minimum required gage of the metal structure is determined with respect to durability, the metal thickness is added to the structural design metal thickness requirements to determine the minimum design thickness.

Adjustment of the predicted metal loss due to abrasion has been incorporated into Figures D12.6.9.5P-1 and D12.6.9.5P-2. A similar relationship has yet to be developed for aluminum and aluminized culverts; however, the minimum predicted metal loss rate of 1 mil/year should be used for the adjustment.

12.6.10.1.2P Reinforced Concrete

A correlation of durability factors to service life of reinforced concrete culverts is presented in Figure D12.6.9.5P-5. Adjustment of the service life to account for abrasion in aluminum culverts is presented in Figure D12.6.9.5P-6. Service life (years-to-poor for culverts with

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sediment) is equal to the service life (years-to-poor, no sediment) multiplied by a sediment factor. Years-to-poor, as defined by the field performance study, occurs when only 50% of the invert thickness remains.

12.6.10.2P Present Worth Analysis

Present worth analysis is a method for adjusting all annual and future costs to a present cost, to permit economic comparison between alternatives. When two or more alternatives are capable of performing the same functions, the more cost-effective alternative will have the least present worth. The present worth method of analysis is restricted to alternatives that perform the same function.

The present worth life-cycle cost for culvert installations shall be determined in accordance with the following relationship (TRB, 1985)

$$LCC = PC + MC \left(\frac{1+I}{1+i} \right)^n - SV \left[\left(\frac{n_p - n}{n_p} \right) \left(\frac{1+I}{1+i} \right)^n \right] \quad (12.6.10.2P-1)$$

where PC is used to compute the initial cost of the culvert, including the cost for culvert materials, backfill and labor costs, and $MC \left[\left(\frac{1+I}{1+i} \right)^n \right]$ is used to compute the present cost for maintenance for year n of the culvert. If a uniform annual maintenance cost is required over the service life of the culvert, the latter term shall be revised as follows:

$$MC \left[\frac{\left(\frac{1+I}{1+i} \right) \left[1 - \left(\frac{1+I}{1+i} \right)^n \right]}{1 - \left(\frac{1+I}{1+i} \right)} \right] \quad (12.6.10.2P-2)$$

The term $SV \left\{ \left[\left(\frac{n_p - n}{n_p} \right) \left(\frac{1+I}{1+i} \right)^n \right] \right\}$ is used to compute the value remaining in the culvert at the end of the anticipated service life of the culvert. The removal cost shall be included in the salvage value term; therefore, the value of this last term could be negative. If the material life is equal to the project design life, the present worth of each alternative is based on a comparison of present or initial costs. When the material life is less than the project design life, alternative comparisons are based on initial costs and replacement or maintenance costs. Similarly, if the material life exceeds the design life, the present worth of the structure is a function of the initial costs with an adjustment for the future value of the structure. Note that PC, MC, and SV values must be in terms of the present worth of that cost.

For life-cycle cost comparisons, the projected inflation and interest rates are difficult to estimate. For preliminary analysis purposes, an average ratio of the inflation to the interest factor (i.e., $\left[\left(\frac{1+I}{1+i} \right) \right]$ of 0.9853 may be used.

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C12.6.10.2P

Because of the range of costs associated with the various alternative methods that can be used to satisfy a particular design, life-cycle cost comparison can be made to determine the alternative which is most economically feasible. The following example may be helpful in understanding the application of life-cycle cost analysis.

Three alternatives are available for improvement of an existing culvert which crosses below a rural route in central Pennsylvania. Alternatives A and B involve replacement of the existing structure. Both alternatives will require maintenance at some time during the service life of the buried structure. Alternative C has an annual maintenance cost for continued use of the existing structure and, therefore, no salvage value. Determine the most economical alternative.

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	Alternatives		
	A	B	C
Present Material and Installation Cost (PC)	\$22,000	\$16,000	\$0
Material Design Life (n_p)	100 years	75 years	50 years
Maintenance Cost (MC)			
Year 30	\$0	\$10,000	\$0
Year 40	\$3,000	\$0	\$0
Annual	\$0	\$0	\$600/year
Salvage Value (SV)	\$7,000	\$3,000	\$0
Service Life (n)	50 years	50 years	50 years
$\frac{\text{Inflation}}{\text{Interest Factor}} \left(\frac{1+I}{1+i} \right)$	0.9853	0.9853	0.9853

$$LCC = PC + MC \left(\frac{1+I}{1+i} \right)^n - SV \left[\left(\frac{n_p - n}{n_p} \right) \left(\frac{1+I}{1+i} \right)^n \right]$$

Alternative A

(a) Step 1

Convert maintenance cost in Year 40 to present worth cost.

$$\$3,000 (0.9853)^{40} = \$1,659.05$$

(b) Step 2

Convert salvage value in Year 50 to present worth cost. (Note that the salvage value includes the structure removal cost.)

$$\$7,000 \left(\frac{100 - 50}{100} \right) (0.9853)^{50} = \$1,669.14$$

$$LCC = \$22,000 + \$1,659.05 - \$1,669.14 = \$21,989.91$$

Alternative B

(a) Step 1

Convert maintenance cost in Year 30 to present worth cost.

$$\$10,000 (0.9853)^{30} = \$6,412.90$$

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(b) Step 2

Convert salvage value in Year 50 to present worth cost. (Note that the salvage value includes the structure removal cost.)

$$\$3,000 \left(\frac{75 - 50}{75} \right) (0.9853)^{50} = \$476.90$$

$$\text{LCC} = \$16,000 + \$6,412.90 - \$476.90 = \$21,936.00$$

Alternative C

(a) Step 1

Convert uniform annual cost to present worth cost.

$$\$600 \left(\frac{0.9853 [1 - (0.9853)^{50}]}{1 - 0.9853} \right) = \$21,037.30$$

(b) Step 2

$$\text{LCC} = \$0 + \$21,037.30 + \$0 = \$21,037.30$$

From the above analysis, Alternative C is the most economical alternate since it requires the least LCC.

12.6.10.3P Other Factors

Various alternates, such as culvert coatings, as discussed in D12.6.9.4P, may be considered to enhance culvert durability. Lining a culvert may also be more effective and cost-efficient than replacement and shall be considered part of the LCC analysis.

12.7 METAL PIPE, PIPE ARCH AND ARCH STRUCTURES**12.7.2 Safety against Structural Failure****12.7.2.7P Standard Designs**

Use Standard Drawing BD-635M entitled "Design Tables for Metal Culverts", and note that it includes consideration for corrosion protection.

Value engineering or alternate design by the contractor should not be permitted, unless it is approved by the District Executive in consultation with the District Bridge Engineer.

It is not required to prepare structure drawings for pipes having an inside span length of less than 8 ft. Thus, the designs under 8 ft. in span are roadway items and shall be included in the construction plan.

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12.7.4 Stiffening Elements for Structural Plate Structures

The following shall supplement A12.7.4.

The maximum allowable spacing for circumferential stiffening elements shall be 54 in.

12.7.5 Construction and Installation

The following shall replace A12.7.5.

The contract documents shall require that construction and installation conform to Publication 408 and Standard Drawing RC-30M.

C12.7.5P

Deflections of metal box culverts are dependent on many factors, including backfill characteristics, compaction, pavement type, vehicle weight and number of load cycles. The use of higher quality backfill, improved backfill compaction and stiff pavements will result in smaller culvert deflections. Live load deflections increase in proportion to increases in the axle load. Deflections will be larger under first loading. After many load applications, deflections are likely to be about one-quarter of those under the first loading of the same load.

12.8 LONG-SPAN STRUCTURAL PLATE STRUCTURES**12.8.2 Service Limit State**

The following shall replace A12.8.2.

The provisions of D12.6.2.1 shall apply.

12.8.3 Safety against Structural Failure**12.8.3.1 Section Properties***12.8.3.1.1 Cross-Section**C12.8.3.1.1*

The following shall supplement AC12.8.3.1.1.

Because no rational design procedure exists for the structural design of long-span structures with acceptable special features in Table A12.8.3.1.1-1, their design is based on the successful performance of completed structures. AASHTO and ASTM technical committees are currently developing recommended design procedures for these structures.

12.8.3.5 Acceptable Special Features*12.8.3.5.2 Reinforcing Ribs*

The following shall supplement A12.8.3.5.2.

When required to satisfy the structural design, reinforcing ribs shall be attached to the structural plate corrugation crown at a bolt spacing of not more than 12 in.

When required only to control structure shape during installation, reinforcing ribs shall be spaced and attached to the corrugated plates as required by the manufacturer and with the approval of the Chief Bridge Engineer.

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12.8.8 Construction and Installation

The following shall supplement A12.8.8.

If approved by the Chief Bridge Engineer, the contract documents shall require that construction and installation of long-span structural plate structures conform to the requirements of Publication 408 and Standard Drawing RC-30M.

12.9 STRUCTURAL PLATE BOX STRUCTURES**12.9.1 General**

The following shall supplement A12.9.1.

Metal box culverts shall be designed with cement concrete spread footings, or a full steel invert.

12.9.2 Loading

The following shall replace A12.9.2.

For live loads, the provisions of A3.6.1 and D3.6.1 shall apply.

C12.9.2

Delete AC12.9.2.

12.9.3 Service Limit State

The following shall supplement A12.9.3.

The provisions of D12.6.2.1 shall apply.

12.9.4 Safety against Structural Failure**12.9.4.2 Moments Due to Factored Loads**

The following shall supplement A12.9.4.2.

The standard dead and live load conditions shall be taken as:

Dead Load: Density of soil backfill,
 $\gamma_s = 0.140$ kcf

Live Load: 40 kip four-wheel single axle
(HS25 loading)
 $A_L = 40$ kip

C12.9.4.2

The following shall supplement AC12.9.4.2.

The designs based on this provision are controlled by the crown and haunch moments. The critical live load position for both crown and haunch moments is at or near the center of the span.

12.9.4.3 Plastic Moment Resistance**C12.9.4.3**

The following shall supplement AC12.9.4.3.

Equations A12.9.4.2-1, A12.9.4.2-2 and A12.9.4.2-3 provide the unfactored dead and live load moments for design. These moments are distributed between the crown and haunch of a box culvert. Theoretically, this distribution could be as unbalanced as 0 to the haunch and 100% to the crown; however, this provision limits distribution to the crown to between 45% and 70%. Analyses, tests and applications have validated the range of distributions prescribed in Tables A12.9.4.3-1 and A12.9.4.3-2. Hence,

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12.9.4.6 Concrete Relieving Slabs

The following shall replace the second paragraph of A12.9.4.6.

The length of the cement concrete relieving slab shall be at least 10 ft. greater than the culvert span and shall project 5 ft. beyond the haunch on each side of the culvert. Slab projections in excess of 1 ft. will result in further reduction in culvert moments due to live load. The relieving slab shall extend across the width subject to traffic loading.

The following shall supplement the third paragraph of A12.9.4.6.

Relieving slab thickness shall be determined using $R_{AL} = 1.05$ for HS25 loading, $R_C = 1.19$ for Class A Cement Concrete, and $t_b = 9$ in. Construction documents shall ensure that granular material is specified under the concrete slab and compacted as per Publication 408, Section 206.3(b)1. Construction documents shall also specify a minimum slab thickness of 13 in., minimum reinforcement of No. 4 bars at 12 in. in both directions, top and bottom, and use of epoxy-coated bars if the slab is under less than 2 ft. of fill.

The following shall supplement A12.9.4.6.

If a cement concrete relieving slab is used immediately above an aluminum culvert, the aluminum shall be physically separated by an inert material to preclude chemical reaction with the concrete.

For construction loading, the relieving slab thickness shall be checked for a 50 kip axle load for self-propelled permit load construction equipment ($R_{AL} = 1.15$).

12.9.5 Construction and Installation

The following shall replace A12.9.5.

The contract documents shall require that construction and installation conform to Publication 408.

plastic moment capacities of crown and haunch shall comply with the ratios of distribution in Tables A12.9.4.3-1 and A12.9.4.3-2.

To account for longitudinal spreading of live loads and resulting reduction in maximum haunch moment. The term R_h is used in equation A12.9.4.3-2.

The specification is structured to permit the user to make adjustments in many of the parameters. However, it is possible to generate a simplified design table for standard conditions. Maximum, median and minimum moments, proportioned in accordance with values found in Tables A12.9.4.3-1 and A12.9.4.3-2, are shown for various span ranges and cover depths. The haunch moment values have been adjusted by the appropriate values for R_h found in Table A12.9.4.3-3.

C12.9.4.6

The following shall supplement AC12.9.4.6.

It is not desirable to locate the concrete relieving slab at top of the roadway as pavement. This type of relieving slab could be inadvertently removed in the future by the maintenance force or under pavement rehabilitation contracts without realizing the structural importance.

The slab adjustment factor, R_t , is based on the results of finite elements analyses comparing stresses in slabs on soil without underlying culverts.

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12.10 REINFORCED CONCRETE PIPE

The following shall replace all provisions of A12.10.

Refer to Appendix H "Pennsylvania Installation Direct Design (PAIDD) for Concrete Pipes" and Standard Drawing BD-636M for structural design criteria and to Publication 280 for manufacturing specifications.

12.11 REINFORCED CONCRETE CAST-IN-PLACE AND PRECAST BOX CULVERTS AND REINFORCED CAST-IN-PLACE ARCHES**12.11.1 General**

The following shall supplement A12.11.1.

Box structures shall be designed using the computer program BXLRFD, entitled LRFD Box Culvert Design and Rating.

The haunch shall be dimensioned to satisfy design, transportation and construction requirements. The haunch dimensions for precast box culverts shall not be less than 6 in. x 6 in.

12.11.2 Loads and Live Load Distribution**12.11.2.1 General**

The following shall replace A12.11.2.1.

Loads and load combinations specified in Tables A3.4.1-1 and D3.4.1.1P-4 shall apply. Live load shall be as specified in A3.6.1.3 and D3.6.1.3. Distribution of wheel loads and concentrated loads for culverts with less than 2 ft. of fill shall be taken as specified in A4.6.2.10 and D4.6.2.10. For traffic traveling parallel to the span, box culverts shall be designed for a single loaded lane with the single lane multiple presence factor applied to the load. Requirements for bottom distribution reinforcement in top slabs of such culverts shall be as specified in A9.7.3.2 for mild steel reinforcement and A5.14.4.1 for prestressed reinforcement.

Edge beams shall be provided as specified in A4.6.2.1.4 as follows:

- At ends of culvert runs where wheel loads travel within 2 ft. from the end of culvert.
- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint.

Distribution of wheel loads to culverts with 2 ft. or more of cover shall be as specified in A3.6.1.2.6.

The dynamic load allowance for buried structures shall conform to A3.6.2.2.

Reinforced concrete box culverts at grade shall be designed for an additional dead load of 0.030 ksf of slab to account for the placement of future wearing surfaces. The

COMMENTARY

C12.10

The following shall replace all provisions of AC12.10. Refer to Appendix H.

C12.11.1

The following shall supplement AC12.11.1.

The designs for precast box culverts in AASHTO M 259M and M 273M are for an HS20 live load and a maximum span length of 12 ft.. For other live load conditions, the designs should be modified.

The documentation for BXLRFD includes narrative on when the haunch is considered or neglected when using that program.

C12.11.2.1

The following shall supplement AC12.11.2.1.

This equivalent strip width policy results in force effects that are generally consistent with the effects computed using the AASHTO Standards Specifications where the strip widths are based on wheel loads rather than axle loads. This policy relies on the interlocking mechanism of the precast units to distribute wheel loads across culvert joints when the equivalent strip width exceeds the segment length.

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thickness of the top slab shall include a 1/2 in. integral wearing surface which shall not be considered in the design of the effective depth of the slab. This additional thickness shall be neglected for slabs incorporating a bituminous wearing surface.

12.11.2.2 Modification of Earth Loads for Soil-Structure Interaction

The following shall replace all provisions of A12.11.2.2.

The total earth load, W_E , on the box section shall be determined as specified in Equation A12.10.2.1-1. The value of B_c shall be taken as specified in Figure 12.11.2.2-1.

For embankment or trench installations, the soil-structure interaction factor, F_e , shall be determined using Equation 12.11.2.2-1.

$$F_e = 1 + 0.20 H/B_c \quad (12.11.2.2-1)$$

where:

H = height of fill over pipe or culvert.

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section, or 1.40 for installations with uncompacted fill along the sides of the box section.

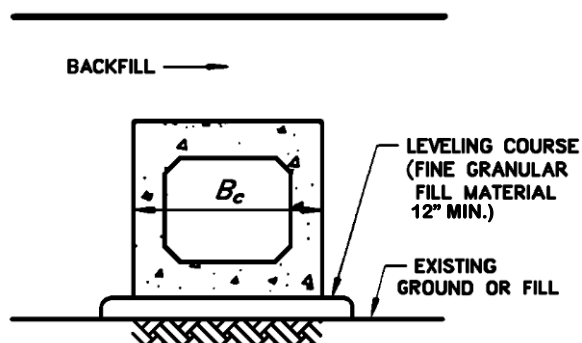


Figure 12.11.2.2-1 - Design Nomenclature for Embankment or Trench Condition Precast Concrete Box Sections

12.11.2.5P Ends of Box Section

The ends of box sections shall be normal to the walls and centerline of the box section. When a beveled end is specified, an edge beam design shall be required.

C12.11.2.5P

The short side of an end segment should be at least 2 ft. in length. The designer must review the detail to ensure proper structural function of the skewed ends.

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12.11.4 Safety against Structural Failure**12.11.4.1 General**

The following shall supplement A12.11.4.1.

If feasible and economical, trussed bars may be used in the design of reinforced concrete box culvert slabs when required to resist diagonal tension. The use of trussed bars shall be approved by the District Bridge Engineer as part of TS&L approval. Stirrups shall not be used.

12.11.4.1.1P Cast-in-Place Arches

Arches of clear normal span up to 15 ft. may be designed with semi-circular or segmental intrados. Larger arches shall be designed with multi-centered intrados derived from an axis conforming as nearly as possible to the equilibrium polygon for full dead load or for full dead load and one-half live load over full span, including lateral earth pressures, to reduce bending moments to a minimum under critical loading conditions.

Culverts on yielding foundation, suitable for either open or tied arch, shall be investigated to determine the most economical type.

12.11.4.4 Minimum Cover for Precast Box Structures

The following shall replace the second and third paragraphs of A12.11.4.4.

The minimum cover shall be in accordance with Standard Drawing BD-632M.

12.11.4.5P Construction and Expansion Joints

Construction joints shall be provided at approximate 30 ft. intervals. Expansion joints (in tied arch and box sections, including the bottom slab and in the ring of open arches) shall be provided at approximate 90 ft. intervals, except as noted subsequently. All joints shall be normal to centerline of culvert.

When appreciable settlement of foundation material is anticipated, and the use of cambered grade for the flow line of a tied arch or box culvert is considered, only construction joints shall be used throughout the entire length of the culvert. Additional longitudinal reinforcing steel (reinforcing steel installed parallel to the centerline of culvert) shall be provided, as required.

For cast-in-place arches, construction joints between the arch ring and footing or the tie slab shall be shown on the drawings.

For precast box culverts, the joints between the box sections shall be sealed with waterproofing sealer. Waterproofing materials shall be in accordance with Publication 408, Section 680, and shall be placed on the top (limits to be determined based on fill height) and the side joints of the box at every joint as per Publication 408,

C12.11.4.1P

When trussed bars are used, the reinforcement and concrete should be placed in a manner that provides for adequate cover and bonding of the reinforcement.

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Section 680 and Standard Drawing BC-788M.

12.11.4.6P Reinforcement Details

For tied arches bars on outside surface of arch shall be bent to the design radius and shall be anchored sufficiently into the bottom of the slab. Main circumferential reinforcement bars for arches shall be placed normal to the centerline of the arch. In skewed end panels, main reinforcement bars shall be cut to fit and shall be anchored properly into the end wall. If appropriate, end panels of skewed arches may be constructed with square ends. The required area of longitudinal reinforcement steel (parallel to the centerline of the culvert) shall be the same as for all shrinkage and temperature reinforcement as specified in A5.10.8 and D5.10.8. However, it shall not be less than No. 4 bars at 12 in. each face. Minimum W4 (0.226 in. diameter) annealed iron wire ties shall be used to prevent buckling of longitudinal bars due to axial thrust in the arch ring. Ties shall be staggered by placement at alternate intersections of longitudinal and transverse bars.

Main circumferential steel reinforcement in cast-in-place box culverts shall be placed normal to the centerline of the culvert. In skewed end panels, main circumferential reinforcement shall be cut to fit and shall be anchored properly into headwalls or edge beams at the top and bottom. If practical, end panels of skewed box culverts may be built with square ends. The required area of longitudinal steel reinforcement (parallel to box culvert centerline) shall be the same as shrinkage and temperature reinforcement specified in A5.10.8 and D5.10.8, except for the top slab of boxes with 2 ft. or less cover where A9.7.3.2 controls the required amount of distribution reinforcement. For main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement shall be used in the middle half of the slab span, and not less than 50% of the specified amount shall be used in the outer quarters of the slab span.

Welded deformed wire fabric for precast box culverts shall be furnished in flat sheets. The center-to-center spacing of main circumferential wire shall not be less than 2 in. and not more than 4 in. (3 in. is preferred).

12.11.4.7P Minimum Thickness

Refer to Standard Drawing BD-633M for minimum thickness requirements for cast-in-place arches and Standard Drawing BD-632M for minimum thickness requirements for cast-in-place and precast box culverts.

12.11.4.8P Minimum Concrete Cover

Refer to D5.12.3 and Standard Drawing BD-632M for minimum concrete cover for steel reinforcement in cast-in-place and precast concrete box culverts.

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12.11.4.9P Multi-Cell Installations

For multi-cell installations of precast box culverts, positive lateral bearing by continuous contact between the sides of adjacent boxes shall be provided by means such as compacted soil fill, granular backfill, grouting or concreting. The method selected shall be shown on the contract drawings.

Details for the joint between cells of multi-cell installations shall be in accordance with Standard Drawing BD-632M.

12.11.5 Construction and Installation

The following shall replace A12.11.5.

The contract documents shall require that construction and installation conform to the requirements of Publication 408, Contract Special Provisions and Standard Drawings RC-11M and RC-12M.

To account for irregularities in culvert excavation and scour, the thickness of the bottom slab shall be increased in accordance with Standard Drawing BD-632M.

12.12 THERMOPLASTIC PIPES**12.12.1 General**

The following shall supplement A12.12.1.

Thermoplastic pipe properties vary with temperature and duration of loading. This aspect shall be considered in the selection of material properties for design.

12.12.2 Service Limit States

The following shall supplement A12.12.2.

The provisions of D12.6.2.1 shall apply.

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12.12.2.1P Deflections

The total horizontal deflection of thermoplastic pipe due to earth and live loads shall be estimated as:

$$\Delta x = \frac{K(D_L W_E + W_L)}{0.149PS + 0.061M_s} \quad (12.12.2.1P-1)$$

where:

Δx = total horizontal deflection (in.)

D_L = deflection lag factor (dim)

K = bedding constant (dim)

W_E = vertical earth load per unit length of pipe (kip/in.)

W_L = vertical live load per unit length of pipe (kip/in.)

PS = pipe stiffness (kip/in./in.)

M_s = modulus of soil reaction (ksi)

For thermoplastic pipes installed in accordance with Standard Drawing RC-30M, the value of M_s shall be taken as 1.5 ksi.

$$PS = \frac{EI}{0.019S^3} \quad (12.12.2.1P-2)$$

where:

E = long-term modulus of elasticity of pipe material at 50 years from Table A12.12.3.3-1 for earth loading and initial modulus of elasticity of pipe material from Table A12.12.3.3-1 for live loading (ksi)

I = minimum moment of inertia of pipe (in.⁴/in.)

S = mean pipe diameter (in.)

ID = inside diameter of pipe (in.)

C = distance from inside surface to neutral axis (in.)

The vertical deflection shall be estimated as

$$\Delta y = \Delta x (ATV) \quad (12.12.2.1P-3)$$

where:

ATV = B. J. Schrock's Correction Factor from Figure 12.12.2.1P-1

COMMENTARY

C12.12.2.1P

If the effects of live load are significant (such as for shallow soil cover less than 5 ft.), deflection estimates should include the effects of live load on the pipe, such as described in ASTM D 3839 using the initial modulus from Table A12.12.3.3-1 for the live load deflection component. The effects of a standard HS20 live load need not be included in Equation 12.12.2.1P-1 for soil covers greater than 5 ft. Where construction vehicles are present, the effects of such vehicles should be evaluated on a case by case basis.

After soil has been initially loaded, it continues to deform (consolidate) with time. The deflection lag factor converts the immediate deflection of the pipe to the deflection of the pipe after many years. Limited investigations concerning this factor have been carried out, but it appears to vary with the type of soil used for the primary zone backfill material and its degree of compaction, native soil characteristic and trench width. For shallow burial depths, a value up to 2.0 appears appropriate when moderate or higher degrees of compaction are attained. With the greater degrees of compaction, the initial pipe deflection is usually very small, and even a slight increase in deflection over a period of time may result in a long-term deflection which can be twice the initial deflection.

The bedding constant reflects the degree of support, provided by the soil at the bottom of the pipe and over which the bottom reaction is distributed. Table C12.12.2.1P-1 gives the recommended values for different types of installation.

Table C12.12.2.1P-1 – Values for Bedding Constant

Type of Installation	Value of K
Shaped bottom with tamped primary pipe zone backfill, 95% Proctor density or greater	0.083
Compacted coarse-grained bedding and primary pipe zone backfill, 70 to 100% relative density	0.083
Shaped bottom, moderately compacted primary pipe zone backfill, 85 to 95% Proctor density	0.103

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The vertical soil load on the pipe may be considered as the weight of the rectangular prism of soil directly above the pipe. The soil prism would have a height equal to the depth of earth cover, a width equal to the pipe outside diameter, and a unit length of 1 in. In the absence of specific soil information, the unit weight of soil may be assumed to be 0.140 kcf.

The values in Table C12.12.2.1P-2 will result in a calculated average deflection so there will be a 50% probability that the actual average deflection will be less than the calculated value. To obtain an estimated average deflection so there will be a 95% probability that the actual average deflection will be less than the calculated value, two procedures are available:

- For $H > 16$ ft. – Use an M_s value equal to 0.75 times the value obtained from Table C12.12.2.1P-2.
- For $H \leq 16$ ft. - Use the M_s value from Table C12.12.2.1P-2 and add the percentage deflections shown as follows to the deflection value so calculated:

Degree of Compaction

Moderate

+1%

High

+0.5%

For pipe installed with a high degree of compaction (95% Proctor, 70% relative density), there is a maximum of 0.5% deflection difference in the average deflections predicted by either procedure.

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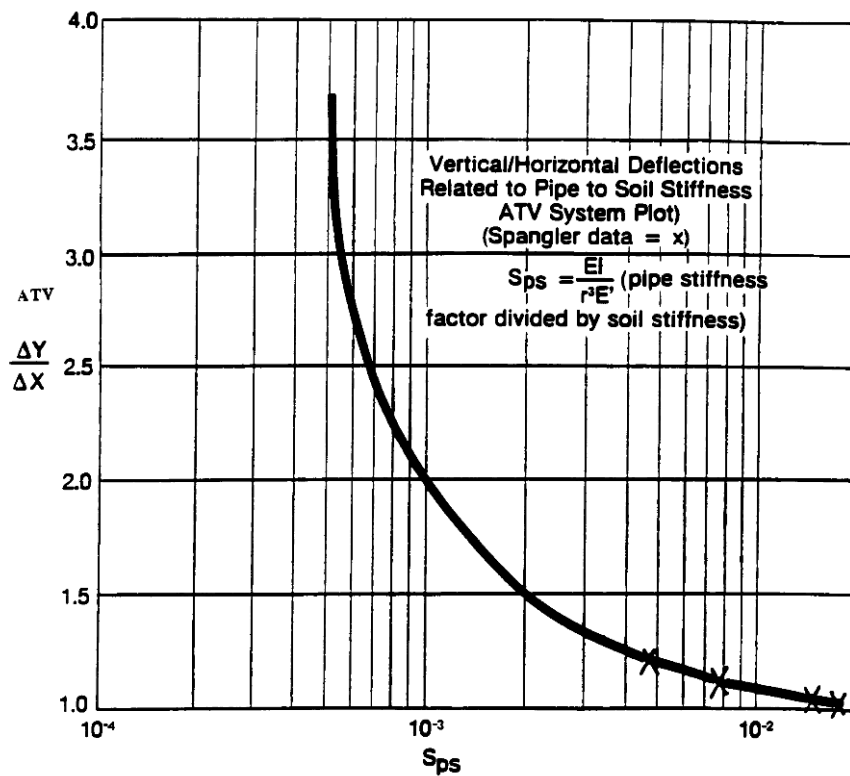


Figure 12.12.2.1P-1 - B. J. Schrock's Correction Factors

Table C12.12.2.1P-2 - Values of Soil Modulus

Soil Type - Primary Pipe Zone Backfill Material	M _s for Degree of Compaction of Pipe-Zone Material	
	Moderate: 85 to 95% Proctor, 40 to 70% Relative Density	High > 95% Proctor, > 70% Relative Density
Fine-grained soils (LL < 50)*: Soil with medium to no plasticity: CL, ML with less than 25% coarse-grained particles	0.4 ksi	1.0 ksi
Fine-grained soils (LL < 50)*: Soils with medium to no plasticity: CL, ML with more than 25% coarse-grained particles	1.0 ksi	2.0 ksi
Coarse-grained soils with fines: GM, GC, SM, SC containing more than 12% fines	1.0 ksi	2.0 ksi
Coarse-grained soils with little or no fines: GW, GP, SW, SP containing less than 12% fines	2.0 ksi	3.0 ksi
Crushed rock	3.0 ksi	3.0 ksi

*LL - Liquid Limit

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12.12.3 Safety against Structural Failure**12.12.3.2 Section Properties**

The following shall supplement A12.12.3.2.

The section properties presented in Appendix A12, Tables A12-11 through A12-13, shall be verified by testing in an independent laboratory, certified by the manufacturer and approved by the Department.

12.12.3.3 Chemical and Mechanical Requirements

The following shall supplement A12.12.3.3.

Use actual long-term (50-year) values of tensile strength (f_u) and Modulus of Elasticity (E) tested by an independent laboratory, certified by the manufacturer, approved by the Department and meeting minimum requirements as shown.

12.12.3.5 Wall Resistance

The following shall supplement A12.12.3.5.

The value of F_u in Equation A12.12.3.5.1-1 shall be the specified minimum tensile strength at 50 years from Table A12.12.3.3-1.

12.12.3.5.2 Buckling

The following shall supplement A12.12.3.5.2.

For side fills conforming to A12.6.6.3, the value of M_s in Equation A12.12.3.5.2-1 shall be based on Sn-95 of Table A12.12.3.4-1.

12.12.4P Submittals

Design submittal shall include detailed section properties with adequate references to standards and tabulated design calculations.

12.13 STEEL TUNNEL LINER PLATE**12.13.1 General**

Delete the second sentence of the first paragraph of A12.13.1.

**12.14 PRECAST REINFORCED CONCRETE
THREE-SIDED STRUCTURES****12.14.1 General****C12.14.1**

Delete AC12.14.1.

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COMMENTARY

12.14.3 Concrete Cover for Reinforcement

The following shall replace A12.14.3.

The minimum concrete cover shall be in accordance with D5.12.3 and D12.11.4.4 for precast box structures.

12.14.5 Design

12.14.5.2 Distribution of Concentrated Load Effects in Top Slab and Sides

The following shall replace A12.14.5.2.

Distribution of wheel loads and concentrated loads for the top slab and sides of three-sided structures with less than 2 ft. of fill shall taken as specified in D12.11.2.1.

Distribution of wheel loads and concentrated loads for the top slab and sides for three sided structures with depths of fill 2 ft. or greater shall be taken as specified in A3.6.1.2.6.

12.15 REFERENCES

The following shall supplement the references in A12.15.

American Association of State Highway and Transportation Officials (AASHTO), Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part I, Specification, 15th ed., Washington, DC, 1990 (see AASHTO Interims for more recent updates)

American Association of State Highway and Transportation Officials (AASHTO), Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II, Tests, 15th ed., Washington, DC, 877 p., 1990 (see AASHTO Interims for more recent updates)

American Iron and Steel Institute (AISI), Handbook of Steel Drainage and Highway Construction Products, Washington, DC, 1983

American Society for Testing and Materials (ASTM), "Steel Piping, Tubing, Fittings", Annual Book of ASTM Standards, Vol. 01.01, ASTM, Philadelphia, PA, 902 p., 1994a

American Society for Testing and Materials (ASTM), "Coated Steel Products", Annual Book of ASTM Standards, Vol. 01.06, ASTM, Philadelphia, PA, 604 p., 1994b

American Society for Testing and Materials (ASTM), "Fasteners", Annual Book of ASTM Standards, Vol. 15.08, ASTM, Philadelphia, PA, 474 p., 1994c

American Society for Testing and Materials (ASTM), "Soil and Rock (I): D420-D4941, Building Stones", Annual Book of ASTM Standards, Vol. 04.08, ASTM, Philadelphia, PA, 978 p., 1994d

American Society for Testing and Materials (ASTM), "Plastic Pipe and Building Products", Annual Book of ASTM Standards, Vol. 08.04, ASTM, Philadelphia, PA, 1236 p., 1994e

American Society of Civil Engineers (ASCE), Design and Construction of Sanitary and Storm Sewers, prepared by a joint committee of the ASCE and the Water Pollution Control Federation (WPCF), ASCE - Manual and Reports of Engineering Practice - No. 37 (WPCF Manual of Practice No. 9), 350 p., 1969

Bakt, B., "Live Load Testing of Soil-Steel Structures", Ontario Ministry of Transportation and Communication, Report SRR-80-4, 1980

Duncan, J. M., and Drawsky, R. H., "Design and Performance of Aluminum Box Culverts", Geotechnical Engineering, Report No. UCB/GT/83-04, University of California, Berkeley, CA, 1983

Kaiser Aluminum, ALUMINUM STRUCTURAL PLATE INCLUDING SUPER PLATE AND LONG SPAN STRUCTURES, DP-437, Edition 8, Kaiser Aluminum & Chemical Company, Oakland, CA, 15 p., 1985

Kaiser Aluminum and Chemical Corporation (Kaiser Aluminum), Aluminum Storm Water Control, DP-103, 8th ed., 1983

Ontario Ministry of Transportation, Ontario Highway Bridge Design Code and Commentary, 3rd Edition, Publication Management Office, Downsview, Ontario, 714 p., 1992

Transportation Research Board (TRB), "Durability of Drainage Pipe", NCHRP Synthesis No. 50, Washington, DC, 1978

Transportation Research Board (TRB), Life-Cycle Cost Analysis of Pavements, NCHRP Synthesis No. 122, Washington, DC, 1985

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

SECTION 13 - RAILINGS

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SPECIFICATIONS

13.1 SCOPE

13.2 DEFINITIONS

The following shall replace the definition of *Crash Testing of Bridge Railings* in A13.2.

Crash Testing of Bridge Railings – Conducting a series of full-scale impact tests of a bridge railing in accordance with the recommended guidelines in AASHTO’s Manual for Assessing Safety Hardware and NCHRP Report 350 in order to evaluate the railing’s strength and safety performance.

13.4 GENERAL

The following shall supplement A13.4.

The bridge railing and transition type chosen during the design field view process shall be taken from the bridge railings given on the BD or BC Standard Drawings.

Other than at deck expansion joints, open joints in the concrete portion of barriers are not permitted unless approved by the Chief Bridge Engineer.

COMMENTARY

C13.1

The following shall replace the second and third paragraphs of AC13.1.

The bridge railing performance need not be identical over the whole highway network. New railing designs should match site needs leading to a multiple test level concept, as described in AASHTO’s Manual for Assessing Safety Hardware and NCHRP Report 350.

All highway safety hardware accepted prior to the adoption of AASHTO’s Manual for Assessing Safety Hardware (MASH), using criteria contained in NCHRP Report 350, may remain in place and may continue to be manufactured and installed. Highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. New highway safety hardware not previously evaluated must utilize MASH for testing and evaluation.

C13.4

The following shall replace the sixth paragraph of AC13.4.

Procedures for testing railing are given in AASHTO’s Manual for Assessing Safety Hardware.

The following shall supplement AC13.4.

The bridge railings given on the BD or BC Standard Drawings have been approved by the FHWA.

Generally, the concrete barrier shown on the Typical Concrete Barrier Detail on Standard Drawing BD-601M will be specified for bridges.

PennDOT has adopted other bridge barrier designs. The PA HT Bridge Barrier (shown on Standard Drawing BD-615M) and the PA Bridge Barrier (shown on Standard Drawing BD-610M) may be used in place of the Department’s standard 42 inch high Typical Concrete Barrier. The 32 in. Alternate Concrete Barrier and the PA Type 10M Bridge Barrier (shown on Standard Drawing BD-617M) may be used only if the guidelines in Design Manual, Part 2, Chapter 12.10 for TL-4 barriers are met.

The barrier shown on BD-609M may be used on precast or cast-in-place reinforced concrete box culverts on non-limited access arterials, collectors, and local roadways.

Structure-related environmental commitments for bridge guide rail shall be carefully considered and justified. Use of weathering steel is permitted only if it is absolutely necessary.

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Also, note that the weathering steel may deteriorate at a faster rate when it is subject to deicing chemicals. Keep all possible options open for the designer to provide a structurally sound and economical bridge guide rail design during the final design phase. Where it is necessary to deviate from the established standards, justifications and special approvals should be well documented.

13.7 TRAFFIC RAILING**13.7.1 Railing System****13.7.1.1 General**

The following shall supplement A13.7.1.1.

Protective fence in accordance with Standard Drawing BC-701M shall be provided on structures with sidewalks over Interstate and other limited access freeways, except where protective barrier is required. Also, provide protective fence on existing bridges, if unsafe incidents have occurred.

In addition, protective fence on structures with sidewalks shall be considered at the following locations:

- On an overpass near a school, playground, or other site where it would be expected that the overpass would be frequently used by children.
- On all overpasses in large urban areas used exclusively by pedestrians and not easily kept under surveillance by police.
- On overpasses with sidewalks, where experience on nearby structures indicates a need for protective fence.

For bridges with sidewalks over electrified railroads, protective barrier shall be used on affected spans or portions of spans. Use the appropriate protective barrier details as shown on Standard Drawings BC-711M, BC-707M, BC-713M or BC-709M for the type of barrier chosen for the structure.

If required by the Railroad or the Public Utility Commission, bridges without sidewalks over electrified railroads may require the appropriate protective barrier as shown on Standard Drawings BC-711M, BC-707M, BC-713M or BC-709M for the type of barrier chosen for the structure.

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13.7.2 Test Level Selection Criteria

The following shall replace the second paragraph of A13.7.2.

Design Manual, Part 2, Chapter 12.10, Railing System Test Level Selection For Bridges provides the criteria for the selection of Test Level for the project. Under any circumstance that Design Manual, Part 2, does not provide criteria for the selection of the Test Level, then Test Level Five, TL-5, shall be used, except when otherwise directed by the Department.

For bridges on Very Low Volume Roads, it is permissible to utilize a reduced Test Level provided the barrier is crash tested and acceptable to the FHWA. The bridge and bridge site must be checked to verify the Test Level used meets AASHTO guidelines.

The following shall replace Table 13.7.2-1 in A13.7.2.

Table 13.7.2-1 – Bridge Railing Test Levels and Crash Test Criteria

	Vehicle Characteristics	Small Automobiles		Pickup Truck	Single-Unit Van Truck	Van-Type Tractor-Trailer		Tractor-Tanker Trailer
NCHRP Report 350	W (kips)	1.55	1.8	4.5	18.0	50.0	80.0	80.0
	B (ft.)	5.5	5.5	6.5	7.5	8.0	8.0	8.0
	G (in.)	22	22	27	49	64	73	81
	Crash angle, θ	20°	20°	25°	15°	15°	15°	15°
	Test Level	Test Speeds (mph)						
	TL-1	30	30	30	N/A	N/A	N/A	N/A
	TL-2	45	45	45	N/A	N/A	N/A	N/A
	TL-3	60	60	60	N/A	N/A	N/A	N/A
	TL-4	60	60	60	50	N/A	N/A	N/A
	TL-5	60	60	60	N/A	N/A	50	N/A
AASHTO MASH	TL-6	60	60	60	N/A	N/A	N/A	50
	W (kips)	2.42	3.3	5.0	22.0	N/A	79.3	79.3
	B (ft.)	5.5	5.5	6.5	7.5	N/A	8.0	8.0
	G (in.)	N/A	N/A	28	63	N/A	73	81
	Crash angle, θ	25°	N/A	25°	15°	N/A	15°	15°
	Test Level	Test Speeds (mph)						
	TL-1	30	N/A	30	N/A	N/A	N/A	N/A
	TL-2	45	N/A	45	N/A	N/A	N/A	N/A
	TL-3	60	N/A	60	N/A	N/A	N/A	N/A
	TL-4	60	N/A	60	55	N/A	N/A	N/A
	TL-5	60	N/A	60	N/A	N/A	50	N/A
	TL-6	60	N/A	60	N/A	N/A	N/A	50

COMMENTARY

C13.7.2

The following shall replace the first sentence of the first paragraph of AC13.7.2.

The six test levels mentioned herein are intended to correspond with the six test levels contained in AASHTO's Manual for Assessing Safety Hardware and NCHRP Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features.

The following shall replace the seventh paragraph of AC13.7.2.

These criteria, including other vehicle characteristics and tolerances, are described in detail in AASHTO's Manual for Assessing Safety Hardware and NCHRP Report 350.

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The following shall supplement AC13.7.2.

The Department recognizes the performance criteria contained in NCHRP Report 350, Recommended Procedures for Safety Performance Evaluation of Highway Features, including criteria for crash tested barrier installations, as evidence of appropriately assigned Test Level designations. The Department's standard 42 inch high Typical Concrete Barrier is designated as a TL-5 barrier and is listed on the FHWA Approved list of Crash Tested Barriers contained in NCHRP Report 350, Appendix B - Update of the 1997 FHWA Bridge Rail Memorandum.

13.7.3 Railing Design**13.7.3.1 General**

The following shall supplement A13.7.3.1.

Table 13.7.3.1-1P lists barrier types, the performance level they were designed for and the location of standard details. The barriers were designed using the method provided in the Appendix to AASHTO, Section 13.

The Vertical Wall Bridge Barrier and Alternate Vertical Wall Bridge Barrier in Table 13.7.3.1-1P is only to be used in rehabilitation projects where providing new substructure safety wings is beyond the scope of work as follows:

- a) for the transition sections at the end of the structure where the structure length is greater than 100 ft. in length and the Typical Concrete Barrier is used; and
- b) for the entire structure when structure length is less than 100 ft.

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Table 13.7.3.1-1P - Test Level for Barrier Type

Barrier Type	TL	Details on Standard Drawing	FHWA acceptance for use on NHS
Typical Concrete Barrier	5	BD-601M, Sheet 2	•
Alternate Concrete Barrier	4	BD-601M, Sheet 2	•
Split Concrete Glare Screen Median Barrier Detail	4	BD-601M, Sheet 2	
Alternate Split Concrete Median Barrier Detail	4	BD-601M, Sheet 2	•
Concrete Glare Screen Median Barrier Detail	4	BD-601M, Sheet 3	
Concrete Median Barrier Detail	4	BD-601M, Sheet 3	
Vertical Wall Bridge Barrier at Alternate Sidewalk Detail	5	BD-601M, Sheet 4	•
PA Bridge Barrier	5	BD-610M	•
PA HT Bridge Barrier	5	BD-615M	•
PA Type 10M Bridge Barrier	4	BD-617M	•
Vertical Wall Bridge Barrier	5	BD-618M	•
Alternate Vertical Wall Bridge Barrier	4	BD-618M	•
Vertical Wall Bridge Barrier for Composite Adjacent Box Beams	4	BD-618M	
Alternate Concrete Barrier for Plank Beams	4	BD-661M, Sheet 4	
Typical Concrete Barrier for Composite Adjacent Box Beams	5	BD-661M, Sheet 4	•
Alternate Concrete Barrier for Composite Adjacent Box Beams	4	BD-661M, Sheet 4	•
Vertical Wall Bridge Barrier for Composite Adjacent Box Beams	5	BD-661M, Sheet 5	•

13.7.3.1.2 New Systems

The following shall replace the second paragraph of A13.7.3.1.2.

The crash test specimen for a railing system may be designed to resist the applied loads in accordance with AASHTO Appendix A13.

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13.7.3.2 Height of Traffic Barrier or Railing

C13.7.3.2

The following shall supplement A13.7.3.2.

The required railing heights are reflected in the BD-601M, BD-610M, BD-615M, BD-617M and BD-618M Standard Drawings.

The following shall replace the first paragraph of AC13.7.3.2.

These heights have been determined as satisfactory through crash tests performed in accordance with NCHRP Report 350 and experience.

13.8 PEDESTRIAN RAILING

13.8.1 Geometry

The following shall supplement A13.8.1.

Unless otherwise directed by the Department, pedestrian railing shall be provided as given in the BC Standard Drawings.

13.9 BICYCLE RAILINGS

13.9.2 Geometry

The following shall replace the first paragraph of A13.9.2.

The height of a bicycle railing shall not be less than 54 in., measured from the top of the riding surface.

13.12 REFERENCES

The following shall supplement A13.12.

AASHTO, 2009, Manual for Assessing Safety Hardware, MASH-1, American Association of State Highway and Transportation Officials, Washington, DC.

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PART 4

VOLUME 1
PART B: DESIGN SPECIFICATIONS

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COMMENTARY

14.1 SCOPE

The following shall supplement A14.1.

The materials, fabrication and installation of the bearings shall be accordance with Publication 408.

14.2 DEFINITIONS**C14.2P**

The following shall supplement A14.2.

Guided Expansion Bearings - Bearings which allow rotation and longitudinal movement in the bearing plane; transverse movement shall be restricted.

In rare cases, some guided bearings may restrict longitudinal movement and allow transverse movement in the bearing plane.

Non-Guided Expansion Bearings - Bearings which allow rotation, longitudinal movement and transverse movement in the bearing plane.

14.3 NOTATION

The following shall supplement A14.3.

A_b = bonded area of rubber

A_r = reduced net bonded area of rubber
 $= A_b (1 - \Delta/B)$

Δ = the shear deflection of the bearing appropriate to the calculation

B = plan dimension in loaded direction of rectangular bearing or diameter of circular bearing

D = superstructure depth from bearing to top of deck (in.) (D14.5.3.2)

d_i = lateral displacement under earthquake loads as specified in D4.7.4.6P

k = material constant

P = maximum vertical load resulting from the Extreme Event I with the γ_p factors equal to 1.0

ϵ_{eq} = shear strain due to d_i , the seismic design displacement

ϵ_{sc} = shear strain due to vertical loads

ϵ_{sh} = shear strain due to maximum horizontal displacement resulting from creep, post-tensioning, shrinkage, and thermal effects computed between the installation temperature and the least favorable extreme temperature

ϵ_{sr} = shear strain due to imposed rotation

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ϵ_u = minimum elongation-at-break of rubber

θ = rotation imposed on bearing

14.4 MOVEMENTS AND LOADS**14.4.2 Design Requirements**

The following shall supplement A14.4.2.

Thermal movements and rotations for all bearings and joints shall be designed using Service-I limit state with $\gamma_{TU}=1$.

C14.4.2

The following shall supplement AC14.4.2.

The Department does not use $\gamma_{TU}=1.2$ as shown in Table A3.4.1-1. Structures in Pennsylvania have been successfully designed using the temperature ranges listed in Table D3.12.2.1-1 under Service-I Limit State with $\gamma_{TU}=1$ and construction tolerances.

14.4.2.2 High Load Multirotational (HLMR) Bearings

The following shall supplement A14.4.2.2.

The service limit state rotation, θ_s , for bearings such as pot bearings, disc bearings, and curved sliding surfaces that may develop hard contact between metal components shall be taken as the sum of:

- The rotations due to all applicable Service I loads;
- The maximum rotation caused by fabrication and installation tolerances, which shall be taken as 0.01 radian unless an approved quality control plan justifies a smaller value; and
- An allowance for uncertainties, which shall be taken as 0.01 radian unless an approved quality control plan justifies a smaller value.

14.4.2.2.1 Pot Bearings and Curved Sliding Surface Bearings

Delete A14.4.2.2.1.

14.4.2.2.2 Disc Bearings

Delete A14.4.2.2.2.

14.5 BRIDGE JOINTS**14.5.1 Requirements****14.5.1.1 General**

The following shall supplement A14.5.1.1.

Superstructure joints shall be in accordance with the provisions in A14.5, D14.5 and the Standard Drawings. Any other type must be evaluated and approved through a process using performance evaluation under heavy truck traffic criteria.

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14.5.2 Selection**14.5.2.1 Number of Joints**

The following shall supplement A14.5.2.1.

The number of expansion joints shall be minimized by designing continuous steel or prestressed concrete structures. Refer to Appendix G for Integral Abutments. For integral abutment details, refer to Standard Drawing BD-667M. For bridge approach slab details, refer to Standard Drawing BD-628M.

14.5.3 Design Requirements**14.5.3.2 Design Movements**

The following shall supplement A14.5.3.2.

Minimum movement classification shall be 3 in., even if less movement is anticipated either at fixed or expansion ends. Round off movement results to the next highest 1/2 in..

The designer shall calculate the end rotation for all joints. End rotation due to applied dead and live loads of girders at the fixed end shall be considered in determining movement classification. End rotation is extremely important at fixed-fixed bearings and to a lesser extent at fixed-expansion bearings. End rotation can be ignored at expansion-expansion bearings, except at skewed joints.

The effect of live load end rotation for a 90° skew fixed end can be roughly approximated by

$$\delta = \frac{4\Delta D}{L} \quad (14.5.3.2-3P)$$

where:

Δ = $L/800$ or $L/1000$ (ft.)

L = length of end span (ft.)

D = superstructure depth from bearing to top of deck (in.)

δ = change in joint opening (in.)

The expansion dams shall accommodate the longitudinal movement due to temperature change, end rotation of superstructure due to deflection, substructure movement and skew effect of the structure. A 1/2 in. contingency allowance (allow 0.66×0.5 in. for low temperature and 0.33×0.5 in. for higher temperature) for joints at abutments, and a 1/4 in. contingency for joints at piers shall be made unless larger movements are anticipated in the foundation report.

A minimum 2 in. movement classification shall be used for the fixed-fixed condition. Opening requirements from end rotation of the fixed-fixed condition must be evaluated.

C14.5.3.2

The following shall supplement AC14.5.3.2.

The end rotation for right angle and skewed bridges is explained below.

- End rotation and joint openings of right angle bridges

At a deck joint on the fixed end of a girder, the increase in joint opening is directly proportional to the amount of end rotation of the girder and to the vertical height from the bearing to the joint. At a similar deck joint on the expansion end of a girder, the amount of joint opening is a function of temperature and span length, as well as the amount of end rotation of the girder and the relative position of the neutral axis with respect to the bearing and the joint. Note that the neutral axis of a girder is unstressed and will not change length under load. However, the compression flange will shorten and the tension flange will lengthen under load.

In the case where the neutral axis is midway between the bearing and the joint, when the girder is loaded the expansion bearing will move away from the fixed bearing by an amount equal to the amount that the joint over the fixed bearing opens, and the joint at the expansion end will remain stationary. If the neutral axis is closer to the joint than to the bearing, when the girder is loaded the expansion bearing will move away from the fixed bearing by an amount greater than the amount that the joint over the fixed bearing opens, and the joint at the expansion end will actually close. If the neutral axis happens to be closer to the bearings than to the joint, the expansion bearing will move a smaller amount than will the joint at the fixed bearing, and the joint at the expansion end will open.

If the guidelines given above are properly considered, the displacement of bearings and joints for normal bridges can be predicted with reasonable accuracy, and provisions can be made for presetting the bearings and joints prior to placing the deck slab so that they will be in proper position in the finished structure. In the past,

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however, the presetting of dams on skewed structures has often resulted in misfits so bad as to require the removal of the dam from the hardened concrete and the resetting of it.

- End rotation and joint openings of skewed bridges

Figure C14.5.3.2-1P represents a schematic plan view of the end of a girder at skewed substructure units. Point A represents a point on a dam directly above the intersection of the centerline of the girder and the centerline of fixed bearings, the girder being undeflected. Point A' represents the position of Point A on the deflected girder as calculated on the basis of the girder rotating about an axis normal to the girder. Point A'' represents the position of Point A on the actual deflected structure. Point E represents a point on a dam directly above the intersection of the centerline of the girder and the centerline of expansion bearings on the same undeflected girder. Point E' represents the position of Point E after a temperature drop that changes the span length by the amount "e". Point E'' represents the position of Point E (due to deflection only) on the actual deflected structure, assuming that the neutral axis of the girder is midway between the bearing and the dam. The location of Point A'' relative to Point A, and Point E'' relative to Point E, is based on the assumption that the ends of the girder will rotate about an axis on the centerline of bearing rather than on an axis normal to the girder. A stiff end diaphragm system validates this assumption.

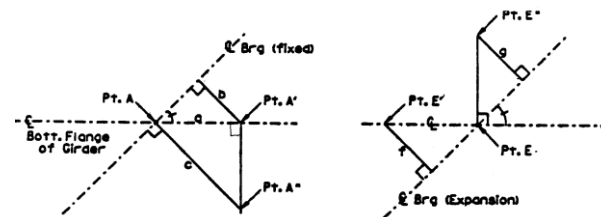


Figure C14.5.3.2-1P - End Rotation and Joint Openings of Skewed Bridges

For thermal movement, e , the change in the expansion dam opening (normal to the dam) will always be

$$f = e \sin (\text{skew angle})$$

For a skewed structure with little or no torsional stiffness, but having an end diaphragm system, the following relationships will hold true for the normal dam openings:

$$c = a / \sin (\text{skew angle})$$

All structures have varying degrees of torsional

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stiffness, which in the case of a skewed structure affects the amount of deflection that will occur under a particular loading condition. The greater the torsional stiffness, the smaller the deflection that will occur. There are no readily available methods of accurately predicting the deflection and subsequent end rotation of a torsionally stiff, severely skewed structure other than sophisticated finite element methods. Therefore, for such cases, a gap (block-out) should be left near the dam in the deck slab when it is poured so that final adjustment of the dam can be made after the major portion of dead load deflection has occurred.

14.5.5 Installation**14.5.5.1 Adjustment**

The following shall supplement A14.5.5.1. Superstructure joints shall be carefully detailed on the design drawings. Provisions shall be made to block out 2 ft. minimum of the deck slab adjacent to a joint so that the joint opening may be adjusted after the deck slab has been placed. The Contractor shall be given the option to eliminate the block-out on 90-degree bridges and for other than tooth dams on skewed bridges, provided the ability to achieve a satisfactory joint in the completed structure is demonstrated with appropriate calculations. Blockouts may also be eliminated if the end rotation due to dead load is less than 1/4 in..

14.5.6 Considerations for Specific Joint Types**14.5.6.10P Selection of Joint Type**

Expansion joint types: If expansion joints are needed, the following shall be specified:

- Strip seals shall be specified for anticipated movements between 0 in. minimum and 4 in. maximum. For construction requirements, refer to Standard Drawing BC-767M.
- Compression seal (unarmored) may be provided for bridges with a design ADTT of 100 or fewer trucks per day. If compression seal is permitted for a bridge, its use shall be specifically indicated either on the bridge plans or in the contract specifications; otherwise, a strip seal shall be used. Compression seals shall be specified to have a 1 in. minimum movement classification.
- Tooth dam with 1/8 in. thick reinforced sheet neoprene trough (50 durometer hardness) shall be specified for movements over 4 in.. Drainage arrangement for the trough, including downspouting, shall be specified as a part of the tooth expansion dam. A minimum of a 2'-6" (but not to exceed shoulder width) plate shall be provided in the gutter area for the safety of bicyclists.

C14.5.6.10P

Table C14.5.6.10P-1 shows the field measurements of various types of proprietary expansion dams. This table can be used to check to ensure a system is available for the computed final design movements.

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Table C14.5.6.10P-1 - Experimentally Determined Perpendicular Movement Capabilities of Evaluated Systems vs. Angle of Crossing

Joint System	Perpendicular Movement Capability (in.)						
	Angle of Crossing (degrees)						
	90	80	70	60	50	40	30
Onflex 25	1.7	1.7	1.7	1.7	1.7	1.7	1.7
Onflex 20	1.8	1.8	1.8	1.8	1.8	1.8	1.6
Pro-Span 2 in. system (low profile)	2.0	2.0	2.0	2.0	1.7	1.5	1.4
Acme 3 in. Strip Seal (AS 300)	3.0	3.0	3.0	2.7	1.9	1.9	1.2
Acme 3 in. Trojan (TR 300)	3.0	3.0	3.0	3.0	2.7	2.0	1.4
Watson Bowman 3 in. system (S 300)	3.0	3.0	3.0	3.0	2.9	2.2	1.4
Acme 4 in. Trojan (TR 400)	3.2	3.2	3.0	2.5	1.5	1.0	0.6
Onflex 40	3.8	3.8	3.8	3.8	3.6	2.8	2.0
Acme 4 in. Strip Seal (AS 400)	4.0	4.0	4.0	2.0	1.7	1.3	1.0
Watson Bowman 4 in. system (S 400)	4.0	4.0	4.0	3.8	2.8	1.8	1.3
Pro-Span 4 in. system (low profile)	4.0	4.0	4.0	3.8	3.2	2.8	2.2
Onflex 45	4.1	4.1	4.0	3.7	3.6	2.8	2.0

Source: F. J. Bashore, et al, "Determination of Allowable Movement Ratings for Various Proprietary Bridge Deck Expansion Joint Devices at Various Skew Angles", Michigan Transportation Commission

14.6 REQUIREMENTS FOR BEARINGS

14.6.1 General

The following shall replace the third paragraph of A14.6.1.

Bearings subject to net uplift at Service I limit state shall be secured by tie-downs, anchorages, or counterweights. The tie-downs, anchorages, or counterweights must be designed to resist the factored net uplift force at Strength I Limit State. If the counterweight extends beyond a distance equal to the depth of the girder from center line of bearing, it should be included in the girder design. Reaction due to counterweights should also be included in the bearing design. Detail the counterweight on the contract drawings.

For additional provisions on bearing anchorages see A14.8.3 and D14.8.3.

14.6.1.1P Fixed and Expansion Bearings

For single span bridges with a span of 35 ft. and less, provide a fixed condition at both abutments. For other single span bridges, arrange the fixed and expansion conditions so that the bridge is expanding uphill, wherever

C14.6.1.1P

Generally, fixed bearings prevent translation in any direction, and expansion bearings permit translation longitudinally, but are fixed against lateral translation. However, these simple arrangements do not always satisfy

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practical.

Where bearings are used to prevent or restrict movement in any direction, a minimum of two such bearings shall be employed on any superstructure unit.

Laminated bearings, known as “Masticord” bearings, are not allowed in Pennsylvania.

14.6.1.2P Multi-Rotational Bearings

All bearings must be capable of permitting rotation in at least one direction. For bridges on tangent alignment, bearing rotation in a single direction is adequate. However, for certain applications such as curved bridges, sharply skewed bridges, and cross girders, the bearings must be able to rotate in more than one direction. In such cases, multi-rotational bearings which utilize an elastomeric material or spherical sliding surfaces shall be used.

The use of multi-rotational bearings shall be indicated where:

- Low profile, high load bearings are required
- Curved or skewed bridges and other similar structures of complex design are required
- Long slender columns or light frames and members exhibit minimum stiffness or rigidity
- The direction of rotation varies
- The direction of rotation cannot be precisely determined
- Self aligning capabilities are required
- Load and rotation eccentricity does not significantly alter the net distribution of stress through the bearing and into the substructure and superstructure
- It is desirable to reduce the moment applied to truss or space frame panels
- Large movements are required

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the required restraints and freedom of movement required for certain types and sizes of bridge superstructures. For instance, thermally-induced movement radiates from a point which is coincident with the center of gravity of the structure. For a bridge on tangent, that point will lie on the longitudinal centerline at the mid-point between the ends of the bridge; regardless of which of the bearings are fixed, the thermal movement will always be parallel to the centerline. However, for a bridge on curved alignment, the center of gravity of the superstructure will be toward the radius of curvature from the centerline; thermal movement will be diagonal to the centerline with respect to that point.

Inspection of Masticord bearings in one Pennsylvania structure revealed completely unsatisfactory performance. The laminated bearings debonded in all layers, i.e., Teflon debonded from stainless steel, Teflon/stainless steel sandwich plate debonded from Masticord material, and the Masticord debonded from the steel sole plate. According to the manufacturer, these bearings are supposed to be particularly forgiving when subjected to non-uniform loading. This has not been experienced in Pennsylvania.

C14.6.1.2P

This section has been revised to include various types of High Load Multi-Rotational Bearings other than just pot bearings. This specification was prepared for the broad range of normal applications and the limits of loads, forces and movements stated. The design and manufacture of multi-rotational bearings relies heavily on the principles of engineering mechanics and extensive practical experience in bearing design and manufacture. Therefore, in special cases in which structural requirements fall outside the limits of this specifications, a bearing manufacturer shall be consulted.

This specification treads a fine line between the need to produce high-quality, long-life bearings and the need to compromise with what is actually possible in design and manufacturer. For this reason, every item in the specification is vital to some aspect of bearing function and cannot be readily changed in the interest of conservativeness or deemed unnecessary and left out.

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- Economical, long life, zero or low maintenance bearings are desirable

Circular neoprene bearings meeting all applicable design requirements may be used as multi-rotational bearings.

Bearings shall be designed for the total movement capacity specified under "Design Movement" in the bearing schedule plus 1 in. additional movement in each direction. Spacing between the guides of the bearing does not require this additional movement capacity. The centerline of all bearing components are symmetrical about the bearing stiffener. The temperature value used for the calculation of longitudinal design movement shall be as per D3.12.2.1.

Only one fixed or guided expansion bearing shall be assumed to resist the sum of all the transverse horizontal forces at each abutment, bent, column, hinge or pier. Seismic forces are an exception as these forces may be resisted by all fixed or guided expansion bearings located at a given substructure unit. Longitudinal loads are resisted only at fixed bearings, and transverse loads are resisted by fixed and guided expansion bearings.

Provide at least two fixed or guided expansion bearings, each able to resist all transverse horizontal forces at each abutment, column, hinge or pier for design redundancy.

The substructure and superstructure shall be designed so that the sole and masonry plates remain rigid under all service conditions in areas around and in contact with the bearings.

14.6.1.2.1P Bearing Schedule

Contract documents shall contain a "bearing schedule" indicating the following information:

- Provide a schedule of all minimum and maximum vertical and horizontal loads for LRFD Load Combinations as shown in Figure 14.6.1.2.1P-1 (this includes all longitudinal and transverse forces, as well as seismic forces) (not applicable for elastomeric bearings).
- Indicate minimum design rotation requirements of the bearing, including a construction tolerance.
- Show magnitude and direction of movements at all bearing support points including thermal, creep and shrinkage movements (see D14.6.1).
- Show the location, quantity and type of each bearing (fixed, expansion, or guided expansion), and the location of all bearing units (an actual bearing layout is preferred or use a bearing framing plan to show this data).

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Using a value for longitudinal design movement equal to twice the amount of contraction provides more movement than theoretically needed. The purpose of using this value is to ensure that adequate movement is provided when the bridge is erected at temperatures greater than 68°F.

C14.6.1.2.1P

Design rotation, movement and other requirements in the bearing schedule shall only refer to the requirements of the structure where the bearings are to be used. The design specifications apply very conservative safety factors to the design requirements.

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- Indicate the nominal stresses and nominal upper and lower bearing contact pressure to be used in the bearing design. They are provided in DM-4 and LRFD Specifications.
- Indicate and properly detail all anchorage details and/or requirements.
- Provide all details and indicate all grades, bevels and slopes at each bearing location.
- Indicate the coefficient of friction used in design of the sliding surfaces. (They should also be used in the substructure design.)
- Highlight any special details needed for earthquake requirements, such as uplift details, temporary attachments, or other requirements.
- Define the surface coating requirements, including the coating specifications and specific surfaces to be coated.
- Field welding of the sole plate to the beam may be done provided a special provision is developed to assure qualified welders and proper welding inspection.
- Show beam seat elevations based on an assumed total bearing thickness detailed in the plans.

SUGGESTED FORMAT FOR LOAD PORTION OF BEARING SCHEDULE

LOAD COMBINATION	FACTORED LOADS (kips)							
	VERTICAL				HORIZONTAL			
	DL		LL+I		TRANSVERSE		LONGITUDINAL	
	MIN	MAX	MIN	MAX	MIN	MAX	MIN	MAX

To the designer:

This table is required for every bearing type (not applicable for elastomeric bearings). Engineering judgment can be used to eliminate groups which obviously will not control the bearing design in order to limit the table size.

Figure 14.6.1.2.1P-1 – Suggested Format for Load Portion of Bearing Schedule

14.6.3 Force Effects Resulting from Restraint of Movement at the Bearing

14.6.3.1 Horizontal Force and Movement

The following shall replace the definition of G and Δ_u for Equation A14.6.3.1-2.

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G = shear modulus of the elastomer (use the highest value of G within the range for elastomer hardness selected, see Table A14.7.5.2-1) (ksi)

Δ_u = factored shear deformation (using the appropriate load factors from A3.4 and D3.4, factor the service value shear deformation, Δ_s , determined from A14.7.5.3.2) (in.)

The following shall replace the last sentence of A14.6.3.1.

For rocker bearings, the factored longitudinal rolling and sliding force is given by

$$H_u = 0.25\gamma(D_1) \frac{r}{R} \quad (14.6.3.1-3P)$$

where:

D_1 = dead load reaction on the rocker (kips)

γ = load factor from A3.4 or D3.4

r = average radius of the web and sole plates or the pin (in.)

R = radius of the rocker (in.)

The following shall supplement A14.6.3.1.

Frictional resistance of bearings slide surfaces shall be excluded when specifying horizontal load requirements.

Minimum horizontal load capacity for fixed or expansion bearings shall be 10 percent of the vertical load capacity.

14.6.4 Fabrication, Installation, Testing and Shipping

The following shall replace A14.6.4.

The provisions for fabrication, installation, testing and shipping of bearings, specified in Publication 408, shall apply

14.6.4.1P Consecutively Fixed Piers

Whenever it is advantageous to the overall design, consecutively fixed piers should be utilized. Generally it will be advantageous for tall, slender piers. However, an analysis should be made, taking into account the stiffness of piers, thermal movements and distribution of horizontal forces. The determination of the number of piers to be consecutively fixed must be based on cost-effectiveness.

When consecutively fixed piers are utilized in a steel bridge design, instructions for jacking the required deflection into the piers for proper positioning of the bearings under the beams shall be shown on the drawings. A table of dimensions shall be included showing the relative distance that each pier must be moved for each 10°F

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temperature variation from the mid-range of the anticipated temperature extremes.

The theoretical fixed point on the bridge, based on the relative stiffness and heights of the piers that are fixed, shall also be shown on the drawings

14.6.4.2P Jacking

Provision shall be made on the superstructure and substructure units to place jacks in order to jack the superstructure for bearing repair or replacement.

The jacking load shall consist of dead load only

14.7 SPECIAL DESIGN PROVISIONS FOR BEARINGS**14.7.1 Metal Rocker and Roller Bearings****14.7.1.1 General**

The following shall replace the third paragraph of A14.7.1.1.

Steel rocker and roller bearings are not permitted on new structures and should be replaced on rehabilitation projects.

Approval from the Chief Bridge Engineer is required when it is desired to leave rocker bearings and/or roller bearings in place on rehabilitation projects.

14.7.2 PTFE Sliding Surfaces**14.7.2.1 PTFE Surface***14.7.2.1.1P Spherical Element - Concave Surface - PTFE*

- The spherical radius shall be determined such that the resulting geometry of the bearing is capable of withstanding the greatest ratio of horizontal load to vertical load under all loading conditions to prevent unseating the concave element
- If required during construction, mechanical safety restraints shall be incorporated to prevent overturning
- Maximum design rotation of the structure itself, plus 0.02 radians, shall be considered in the bearing design to prevent overturning or uplift
- Calculations showing the determination of the radius shall be submitted for approval
- The concave surface shall face down

Refer to Figure 14.7.2.1.1P-1 for spherical bearings.

C14.7.1.1

Delete the third paragraph of AC14.7.1.1.

The following shall supplement AC14.7.1.1.

Figure PP5.5.2.8.2-1 indicates the bearing types to be replaced, and Figure PP5.5.2.8.2-2 indicates schematic examples of bearing replacement details and retrofit detail, when approved by the Chief Bridge Engineer.

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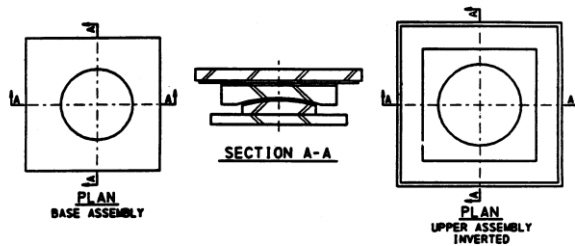


Figure 14.7.2.1.1P-1 - Spherical Bearings

14.7.2.2 Mating Surface

The following shall supplement A14.7.2.2.

For PTFE/Stainless and Bronze/Stainless sliding surfaces, the stainless surface shall be one of the following:

- ASTM A 240/A 240M, Type 304, 0.060 to 0.090 in. thick with a 20 μ -in. rms finish or less.
- Solid stainless steel ASTM A 240/A 240M, Type 304 or 304L, with a 20 μ -in. rms finish or less.
- Stainless steel weld overlay a minimum of 3/32 in. thick with a 20 μ -in. rms finish or less

14.7.2.3 Minimum Thickness**14.7.2.3.1 PTFE**

The following shall supplement A14.7.2.3.1.

PTFE fabric shall be in accordance with ASTM D 1777.

Any edges other than the selvage shall be oversown or recessed so that no cut fabric edges are exposed.

14.7.2.3.2 Stainless Steel Mating Surface

The following shall supplement A14.7.2.3.2.

The stainless steel surface shall cover the mating surface in all operating positions plus 1 in. in each direction of movement or extend to the end of sole plate whichever is greater.

Stainless steel sliding surfaces shall face down.

Stainless steel welded overlay shall be a minimum of 3/32 in. thick after welding, grinding, and polishing, and produced using Type 309L electrodes.

Guided members shall have their contact area within the guide bars in all operating positions. Guiding off the fixed base or any extensions of it is not permitted.

14.7.2.5 Coefficient of Friction

The following shall supplement A14.7.2.5.

Friction coefficients must be used in conjunction with the substructure design.

Design friction factors shall correspond to the type of PTFE specified.

C14.7.2.5

The following shall supplement AC14.7.2.5.

Pot bearing designs taken from BD-613M use a coefficient of friction equal to 0.04 between the PTFE and

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Design friction factors given in Table A14.7.2.5-1 may be used for pot bearings if a 8 μ -in. rms surface finish is specified for the mating surface. In using Table A14.7.2.5-1, a temperature of -13°F shall be assumed.

14.7.3 Bearings with Curved Sliding Surfaces**14.7.3.1 General**

The following shall supplement A14.7.3.1.

Complete design calculations must be provided by the designer for all aspects of spherical bearings. For high load multi-rotational bearing design plan presentation, see D14.6.1.2.1P. Bearings shall be designed for the temperature range found in Table D3.12.2.1-1.

These provisions are directed primarily toward spherical or cylindrical bearings with bronze or PTFE sliding surfaces.

The minimum center thickness of concave spherical surfaces must be at least 3/4 in., and the minimum vertical clearance between the rotating and non-rotating parts is given by Equations 14.7.3.1-1P or 14.7.3.1-2P.

For rectangular spherical or curved bearings:

$$c = 0.7D\theta_u + 0.125 \quad (14.7.3.1-1P)$$

For round spherical or round bearings:

$$c = 0.5D\theta_u + 0.125 \quad (14.7.3.1-2P)$$

where:

D = diameter of the projection of the loaded surface of the bearing in the horizontal plane (in.)

θ_u = design rotation from D14.4.2.2 (rad)

Minimum edge and center thicknesses shall be no less than:

- OD x 0.06 for bearings directly on concrete
- OD x 0.045 for bearings on steel masonry plates
- 1/2 in.

14.7.3.3 Resistance to Lateral Load

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stainless steel assuming the type of PTFE is unfilled, dimpled and lubricated.

A temperature of -13°F is the closest temperature equating to the cold end of the DM-4's design temperature range for structures.

C14.7.3.1

Delete AC14.7.3.1.

The first two paragraphs of AC14.7.3.1 have been moved into the specification portion of D14.7.3.1. This third paragraph has been replaced by some Department requirements and given in the last paragraph of D14.7.3.1.

C14.7.3.3

The following shall supplement AC14.7.3.3.

To prevent the accumulation of debris, the concave surface of the bearing shall face down.

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14.7.4 Pot Bearings**14.7.4.1 General**

The following shall supplement A14.7.4.1.

BD-613M shall be used for pot bearings with vertical service loads between 200 kips and 1500 kips with 0.03 radians maximum rotation. If the standard is not used, complete design calculations must be provided prior to submission of shop drawings on all projects for all aspects of pot bearings, see A14.6.1 and D14.6.1.

Pot bearings are not stiff against bending in their plane. A sole plate, beveled if necessary, on top and a masonry plate at the bottom of the bearing shall be provided. Pot bearings have a limited capacity of rotation; they shall not be mixed with other types of bearings at common superstructure and substructure units.

For high load multi-rotational bearing design plan presentation, see D14.6.1.2.1P.

As a part of final design, designers are responsible for the design of all aspects of the pot bearing as well as the pot base thickness. These items are all included in BD-613M. If the standard is not used, these items must be designed:

- pot diameter
- pot wall thickness and height
- piston shaft diameter (Min.), piston bottom diameter (Min.), and sealing rings
- masonry plate size and thickness
- sole plate size and thickness
- guide plate thickness, length and width (Min.); piston to guide plate weld size (Min.)
- sliding surface dimensions; length of Teflon strip and width (Min.); number of flat head socket screws
- all connection requirements, including weld sizes and number and size of bolts, cap screens and anchor bolts

14.7.4.1.1P Types of Pot Bearings

There are two types of pot bearing designs allowed in Pennsylvania:

- Pot facing up with plates attached to the piston and guided by the edge of the sliding plate. Piston is to be self-aligning to direction of movement, but guides are to be carefully aligned. This pot bearing must be designed for PennDOT structures, unless approved otherwise by the Chief Bridge Engineer at the TS&L stage.
- Design as specified above, but with double elastomeric pad for greater rotational capacity

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14.7.4.2 Materials

The following shall replace the first paragraph of A14.7.4.2.

The elastomeric disc shall be made from a compound based on virgin natural rubber or virgin neoprene conforming to the requirements of Publication 408.

14.7.4.4 Elastomeric Disc

Delete the last sentence of the second paragraph of A14.7.4.4

C14.7.4.4

Delete the last sentence of the second paragraph of AC14.7.4.4.

14.7.4.5 Sealing Rings*14.7.4.5.3 Rings with Circular Cross-Sections*

The following shall replace A14.7.4.5.3.

Sealing rings with a circular cross-section shall have the following minimum diameters:

- Up to 125 kips - 5/32 in.
- Up to 400 kips - 3/16 in.
- Up to 800 kips - 5/16 in.
- Up to 1400 kips - 3/8 in.
- Up to 5000 kips - 1/2 in.

14.7.4.6 Pot

The following shall replace the second and third paragraphs of A14.7.4.6.

The minimum thickness of a base bearing directly against concrete or grout shall satisfy:

- $t_{\text{base}} \geq 0.06 D_p$ and (14.7.4.6-1)
- $t_{\text{base}} \geq 3/4$ in. (14.7.4.6-2)

The thickness of a base bearing directly on steel girders or load distribution plates shall satisfy:

- $t_{\text{base}} \geq 0.045 D_p$ and (14.7.4.6-3)
- $t_{\text{base}} \geq 1/2$ in. (14.7.4.6-4)

The following shall supplement A14.7.4.6.

Pot inside diameter shall be the same as the elastomeric disc.

If BD-613M is not used, the minimum pot wall thickness shall be determined by analyzing horizontal loads, internal elastomer pressure, and piston force due to friction, shear, bending, and tension, but shall not be less than 3/4 in..

Pots shall be connected to the masonry plate by either setting the pot in a recess designed for horizontal loads or by welding the pot to the masonry plate. If the pot is set in a recess proper drainage is required.

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14.7.4.7 Piston**C14.7.4.7**

The following shall replace the third paragraph of A14.7.4.7

Delete the third paragraph of AC14.7.4.7.

Where a mechanical device is used to connect the superstructure to the substructure, it shall be designed to resist the greater of H_s or 10% of the maximum vertical load at the service limit state at that location.

The following shall replace the fourth paragraph of A14.7.4.7 through equation A14.7.4.7-3.

Pot bearings subjected to lateral loads shall be proportioned to satisfy:

$$t_w \geq \sqrt{\frac{40H_s \theta_s}{F_y}} \quad (14.7.4.7-1)$$

Pot bearings that transfer load through the piston shall satisfy:

$$w \geq \frac{2.5H_s}{D_p F_y} \quad (14.7.4.7-2)$$

$$w \geq 0.125 \text{ in.} \quad (14.7.4.7-3)$$

where:

H_s = horizontal service or extreme event I load on the bearing (kips)

θ_s = design rotation from D14.4.2.2 (rad)

F_y = yield strength of steel (ksi)

D_p = internal diameter of pot (in.)

w = height of piston rim (in.)

t_w = pot wall thickness (in.)

The following shall supplement A14.7.4.7.

Piston thickness shall be sufficient to provide clearance between the top of the pot and the sliding surface above as follows:

- For square pots, clearance = (Design Rotation (radians)) $\times$ (0.7 $\times$ Pot side (in.)) + 0.12 $\geq$ 0.2 in.
- For round pots, clearance = (Design Rotation (radians)) $\times$ (Pot OD (in.) / 2) + 0.12 $\geq$ 0.2 in.

Where the seal is wholly within the piston thickness, pistons for round seals shall have the lower corner chamfered at 45° for a depth equal to 1.7 times the diameter of the seal and 1.2 times the diameter where the seal extends into the elastomer. Refer to Figures 14.7.4.7-1P and 14.7.4.7-2P.

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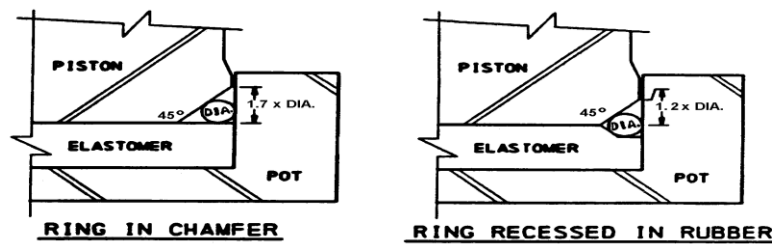


Figure 14.7.4.7-1P - Piston Chamfer for Sealing Rings

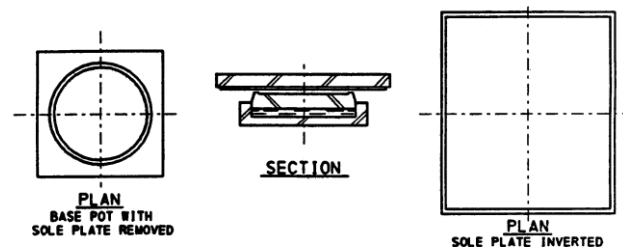


Figure 14.7.4.7-2P - Pot Bearings

14.7.5 Steel Reinforced Elastomeric Bearings – Method B**14.7.5.1 General**

The following shall supplement A14.7.5.1.

The Department has decided to use Method A which has been incorporated in D14.7.6. The Department has selected this method because it has successfully been used in the past and it does not require the additional testing and quality control of bearings designed by Method B.

Elastomeric bearings utilized in implementing isolation design shall be designed by the procedures and specifications given in D14.7.6.3.10P and D14.7.6.3.11P.

Circular neoprene bearings may be used as multi-rotational bearings on skewed bridges.

14.7.5.2 Material Properties

The following shall replace the second paragraph of A14.7.5.2.

The elastomer shall have a shear modulus between 0.080 and 0.189 ksi and a nominal hardness between 50 and 60 on the Shore A scale, and shall conform to the requirements of Publication 408, Section 1113. Use a nominal hardness of 50 unless approved otherwise.

Delete the third paragraph of A14.7.5.2.

The following shall replace the first sentence of the seventh paragraph of A14.7.5.2.

Bearings shall be made from low temperature elastomer grade 3 unless otherwise approved by the Chief Bridge Engineer.

C14.7.5.1

Delete the second paragraph of AC14.7.5.1.

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14.7.5.3 Design Requirements*14.7.5.3.1 Scope*

Delete A14.7.5.3.1.

C14.7.5.3.1

Delete AC14.7.5.3.1.

14.7.5.3.2 Shear Deformations

The following shall supplement A14.7.5.3.2.

For expansion bearings, Δ_s shall include longitudinal movement due to end rotation of beams. A procedure for computing longitudinal movement due to end rotation is provided in the LRFD Bearing Pad Program Manual.

For prestressed concrete bridges, the time when bearing fixity occurs in the construction sequence shall be considered. The construction plans shall indicate a construction sequencing for bearing fixity if a special sequencing is assumed in the design.

Longitudinal movement due to end rotation and beam lengthening for all dead loads (except loads to be applied in the future, e.g., future wearing surface and utility loads) may be eliminated from the pad design for permanent conditions provided that the bridge is jacked after dead loads are applied and the shear deformation in the bearing pads is relieved. The pad should be checked to verify that design criteria is met for the temporary condition prior to release of the dead load shear deformation. A temperature change of 35° F shall be used to check for temporary conditions. The construction plans should clearly indicate where and when jacking is required, and provision for jacking points must be included in the design and detailing of the superstructure and substructure. The jacking forces should also be specified.

C14.7.5.3.2

The following shall replace AC14.7.5.3.2.

The inclusion of applicable shear deformations with the temperature ranges specified in D3.12.2.1 allow contractors to erect beams at expected construction condition temperatures.

Consideration of creep and shrinkage deflection as it was previously computed can be eliminated because its direction of movement is opposite from the longitudinal movement due to end rotation. The magnitude of longitudinal movement due to end rotation will be greater than the previously set value of 0.25 in. for creep and shrinkage and it will thus control.

Longitudinal movement due to end rotation and beam lengthening may result in undesignable or excessively thick pads, especially with steel beam bridges. The requirement for jacking the bridge will result in additional cost and construction time, but the reduction in the longitudinal movement for pad design may eliminate the need to use a more costly bearing type. The designer should evaluate the cost benefits and construction time effects for individual projects when jacking is proposed.

14.7.5.3.3 Combined Compression, Rotation and Shear

Delete A14.7.5.3.3.

C14.7.5.3.3

Delete AC14.7.5.3.3.

14.7.5.3.4 Stability of Elastomeric Bearings

Delete A14.7.5.3.4.

C14.7.5.3.4

Delete AC14.7.5.3.4.

14.7.5.3.5 Reinforcement

The following shall replace A14.7.5.3.5.

Fabric reinforcement shall not be used.

The thickness of the steel reinforcement, h_s , shall satisfy:

- at the service limit state:

$$h_s \geq \frac{h_{avg}(1.7\text{ksi})}{0.55F_y} \quad (14.7.5.3.5-1)$$

C14.7.5.3.5

The following shall replace AC14.7.5.3.5.

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- at the fatigue limit state:

$$h_s \geq \frac{2h_{avg}\sigma_L}{\Delta F_{TH}} \quad (14.7.5.3.5-2)$$

- at PennDOT minimum:

$$h_s \geq 1/8 \text{ in.}$$

where:

h_{avg} = the average thickness of the two layers of the elastomer bonded to the reinforcement (in.)

F_y = yield strength of steel reinforcement (ksi)

ΔF_{TH} = constant amplitude fatigue threshold for Category A as specified in Table A6.6.1.2.5-3 (ksi)

σ_L = service average compressive stress due to live load (ksi)

If holes exist in the reinforcement, the minimum thickness calculated by Equations 14.7.5.3.5-1 or 14.7.5.3.5-2 shall be increased by a factor equal to twice the gross width divided by the net width.

The thickness of the steel reinforcement that is shown on the contract drawings shall be a thickness that is a manufactured steel plate size

14.7.5.3.6 Compressive Deflection

The following shall replace the sixth paragraph of A14.7.5.3.6.

Values for ε_{Li} or ε_{di} shall be obtained from design aids based on tests such as presented in Figure AC14.7.6.3.3-1 by testing or by an approved analysis method. Equations for ε_{Li} or ε_{di} given in DC14.7.5.3.6, may also be used. Figure AC14.7.6.3.3-1 is for internal layers of reinforced bearings. The effects of creep of the elastomer shall be added to the instantaneous deflection when considering long-term deflections. They shall be computed from information relevant to the elastomeric compound used if it is available. If not, the material properties given in A14.7.5.2 shall be used.

The Department has had problems in the past where a steel reinforcement thickness was used which was slightly less than or greater than the steel reinforcement thickness which was specified on the contract drawings.

C14.7.5.3.6

Delete the third paragraph of AC14.7.5.3.6.

The following shall supplement AC14.7.5.3.6.

A second order polynomial equation for ε_{Li} or ε_{di} was developed for use in the PennDOT bearing design computer program from Figure AC14.7.6.3.3-1 for each shape factor of 3, 4, 5, 6, 9 and 12, and for both 50 and 60 durometer elastomer. The basic equation, for instantaneous compressive strain, is:

$$\varepsilon_{Li} \text{ or } \varepsilon_{di} = A\sigma^2 + B\sigma \quad (C14.7.5.3.6-1)$$

where:

σ = average compressive load due to the load being investigated (ksi)

A = a constant given in Table C14.7.5.3.6-1P (1/ksi²)

B = a constant given in Table C14.7.5.3.6-1P (1/ksi)

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Table C14.7.5.3.6-1P – Constants for Strain Equation

Shape Factor	50 Durometer		60 Durometer	
	A	B	A	B
3	-18.00	22.7	-7.30	15.9
4	-4.20	12.3	-2.80	10.2
5	-1.40	8.4	-1.60	7.3
6	-0.69	5.9	-0.90	5.3
9	-0.70	5.2	-0.78	4.6
12	-1.00	5.0	-0.99	4.5

The PennDOT bearing design computer program will use a minimum shape factor of 3.

The maximum relative compressive deflection should be less than 3/4 in.. The method for relative compressive deflection computations for end joints and pier joints is given below.

Relative deflection at end joints is the sum of creep deflection and live load deflection.

Relative deflection at pier joints shall be the greater of the followings:

- (1) The difference of the sum of the dead load deflection, creep deflection and live load deflection on the right side of the joint, and the sum of the dead load deflection and creep deflection on the left side of the joint.
- (2) The difference of the sum of the dead load deflection, creep deflection and live load deflection on the left side of the joint, and the sum of dead load deflection and creep deflection on the right side of the joint.

14.7.5.3.7 Seismic and Other Extreme Event Provisions

The following shall replace the fourth sentence of A14.7.5.3.7.

Anchor bolts shall be designed for the effects of bending and shear, as specified in A14.8.3.1 and D14.8.3.1, for seismic and other extreme event loads as specified in A14.6.5.3.

14.7.6 Elastomeric Pads and Steel Reinforced Elastomeric Bearings - Method A

14.7.6.1 General

The following shall replace A14.7.6.1.

The provisions of this article apply to the design of plain elastomeric pads, PEP and steel reinforced elastomeric bearings.

Pads reinforced with discrete layers of fiberglass, FGP,

C14.7.6.1

Delete AC14.7.6.1.

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and pads reinforced with closely spaced layers of cotton duck, CDP, shall not be used in elastomeric bearings. However, these two materials may be used as a bedding material between the top surface of the pier or abutment concrete and the bearing base plate.

The shape factor for pads and steel reinforced elastomeric bearings covered by this article is determined as specified in A14.7.5.1.

14.7.6.2 Material Properties

The following shall replace A14.7.6.2.

The material properties shall be as given in A14.7.5.2 and D14.7.5.2.

14.7.6.3 Design Requirements*14.7.6.3.1 Scope*

The following shall replace A14.7.6.3.1.

Bearings designed by provisions herein do not require any additional testing other than that required in Publication 408.

14.7.6.3.1aP General Size and Construction Requirements

Reinforced elastomeric bearings shall be reinforced using integrally bonded steel reinforcement. Fabric reinforcement is not permitted.

Plain pad thickness shall be a minimum of 3/4 in. and a maximum of 1 1/4 in..

Laminated pads are most efficiently designed by placing shims near the top and bottom extremities of the pads, which are provided with a bonded neoprene cover of 1/4 in.. Inner layer thickness of neoprene may be specified from 3/8 in. to 5/8 in.. Requirements for minimum shim thickness are given in D14.7.5.3.5.

COMMENTARY

C14.7.6.3.1

Delete AC14.7.6.3.1.

C14.7.6.3.1aP

Shimmed neoprene expansion bearings shall generally not exceed a total thickness of 6 in.. The minimum thickness of shimmed neoprene bearings shall generally be not less than 2 in.. This minimum thickness will provide increased rotational tolerance for spans less than 100 ft.

A relatively new method to align the shims during the vulcanizing of laminated shim neoprene bearing pads has been developed. Two companies have been approved by the Department to use this method. A brief description of this method is given below.

Instead of using several pins around the perimeter of the pad to hold the shims in place during the pad manufacturing process, a single 1.5 in. x 0.5 in. rounded slot in the center of the pad, oriented longitudinally, may be used to accommodate a steel bar which holds and aligns the shims during vulcanizing.

Instead of exhibiting small slotted indentations around the perimeter of the completed pads, typical of laminated pads using the old manufacturing process, the new pads will show 1.5 in. x 0.5 in. "plugged" rounded slot in the center of the finished bearing.

Laminated shims neoprene bearing pads exhibiting either characteristics, slotted indentations on the perimeter or plugged slotted rounded holes in the center, oriented longitudinally, will be acceptable. The plug, or neoprene extrusion, must be of the same hardness as the pad, must be installed with lubricant adhesive, and must provide a tight fit. There is no allowance for a minus tolerance on the size of the plug.

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14.7.6.3.2 *Compressive Stress*

C14.7.6.3.2

The following shall replace A14.7.6.3.2.

Unless shear deformation is prevented, use the lower value of shear modulus G when computing the allowable compressive stress. In any elastomeric bearing layer, the average compressive stress at the Service I Limit State shall satisfy:

$$\sigma_c \leq GS_i \quad (14.7.6.3.2-1)$$

and

$$\sigma_c \leq 1.25 \text{ ksi for steel-reinforced elastomeric bearings} \quad (14.7.6.3.2-2)$$

or

$$\sigma_c \leq 0.8 \text{ ksi for plain elastomeric bearings} \quad (14.7.6.3.2-3)$$

where:

G = shear modulus of elastomeric pad (ksi)

S_i = shape factor given in A14.7.5.1

Allowable compressive stress may be increased for laminated pads by 10% where shear deformation is prevented.

The minimum average compressive stress under DL only shall satisfy:

$$\sigma_c > 0.1 \text{ ksi} \quad (14.7.6.3.2-4)$$

14.7.6.3.3 *Compressive Deflection*

C14.7.6.3.3

The following shall replace A14.7.6.3.3.

The provisions of A14.7.5.3.6 and D14.7.5.3.6 shall apply.

Delete AC14.7.6.3.3.

14.7.6.3.4 *Shear*

C14.7.6.3.4

The following shall replace A14.7.6.3.4.

The provisions of A14.7.5.3.2 and D14.7.5.3.2 shall apply.

Delete AC14.7.6.3.4

14.7.6.3.5 *Rotation*

C14.7.6.3.5

The following shall replace all provisions of A14.7.6.3.5.

The rotational deformations about each axis shall be taken as the maximum possible rotation between the top and bottom of the bearing caused by initial lack of parallelism and girder end rotation. They shall be limited by:

- for rectangular bearings

$$\theta_{sx} \leq \frac{2\delta}{L} \quad (14.7.6.3.5-1)$$

and

The following shall replace all provisions of AC14.7.6.3.5.

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$$\theta_{sz} \leq \frac{2\delta}{W} \quad (14.7.6.3.5-2)$$

- for circular bearings

$$\sqrt{\theta_{sx}^2 + \theta_{sz}^2} \leq \frac{2\delta}{D} s \quad (14.7.6.3.5-3)$$

where:

L = length of a rectangular bearing (measured along the length of the beam) (in.)

W = width of a rectangular bearing (measured along the width of the beam) (in.)

D = diameter of a circular bearing (in.)

δ = instantaneous deflection specified in A14.7.5.3.6 (in.)

θ_{sx} = maximum rotation between the top and bottom bearing about the transverse axis (rad)

θ_{sz} = maximum rotation between the top and bottom bearing about the longitudinal axis (rad)

Live load rotation shall be based on the same load condition used for live load deflection which is given in D2.5.2.6.2 and D3.6.1.3.2.

If the live load deflection is known, the live load rotation may be estimated by:

$$\theta_{LL} = \frac{16}{5} \frac{\delta_{LL}}{L_s} \quad (14.7.6.3.5-4)$$

where:

δ_{LL} = live load deflection (in.)

L_s = span length (in.)

In lieu of Equation 14.7.6.3.5-4, a more exact method may be used to compute live load rotation.

For spans under 100 ft., an additional 0.003 radian of rotation about the transverse axis of the pad shall be considered for construction tolerance.

For spans equal to or over 100 ft., an additional 0.005 radian of rotation about the transverse and longitudinal axes of the pad shall be considered for construction tolerance.

The rotational tolerance shall be applied when checking for rotational adequacy under dead load and dead load plus live load conditions. The dead load shall be computed without future wearing surface, utilities, or other non-permanent dead load.

Equation 14.7.6.3.5-4 is based on an assumed parabolic shape which is associated with dead and live load deflections.

Rotational tolerance is added to account for permissible geometric deviations that occur in superstructure and substructure elements during fabrication and construction. The specified design tolerances do not account for the combined fabrication and construction tolerances that are

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Where the specified construction tolerances result in undesignable pads or excessively thick pads, the construction tolerances may be reduced. For spans under 100 ft., the tolerance about the transverse axis of the pad may be reduced. For spans equal to or greater than 100 ft., the tolerance about the longitudinal axis of the pad may be reduced.

The tolerance values used for the bearing pad design shall be shown on the construction plans.

For steel beam bridges, assess the potential for a lift-off condition (or gap between the bottom of beam sole plate and top of the bearing pad) when the beam is initially set on the pad. Lift-off can be expected when the end rotation occurring for the temporary condition exceeds the rotational capacity as computed using Equations 14.7.6.3.5-1, 14.7.6.3.5-2 and 14.7.6.3.5-3 with beam dead load only. If a lift-off condition is expected, provide the following note on the construction plans:

“A gap between the bottom of sole plate and top of neoprene bearing pad is expected to be present at (Abutment 1, Pier 1, etc.) when the beams are placed on the pads. The gap is expected to close after all dead loads are placed on the bridge.”

Where a temporary lift-off condition is expected at pier bearings of continuous steel beam bridges, the location of the lift-off (ahead or back stations) should be identified.

14.7.6.3.6 *Stability*

The following shall replace A14.7.6.3.6.

To ensure bearing stability, the total thickness, H , of the bearing shall be limited by the smallest of:

- for plain rectangular bearings

$$H \leq L/5 \quad (14.7.6.3.6-1P)$$
 or

$$H \leq W/5 \quad (14.7.6.3.6-2 P)$$
- for plain circular bearings

$$H \leq D/6 \quad (14.7.6.3.6-3 P)$$
- for reinforced rectangular bearings

$$H \leq L/3 \quad (14.7.6.3.6-4 P)$$
 or

$$H \leq W/3 \quad (14.7.6.3.6-5 P)$$
- for reinforced circular bearings

$$H \leq D/4 \quad (14.7.6.3.6-6 P)$$
- for all bearing types

$$H \leq 8 \text{ in.} \quad (14.7.6.3.6-7 P)$$

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permitted in Publication 408. To do so would result in a large percentage of bridges where neoprene bearing pads would be undesignable.

A parametric study conducted on single span prestressed concrete I-beam, spread box beam, and adjacent box beam bridges resulted in undesignable neoprene pads for some conditions when using the full construction tolerance values. The majority of instances occurred in adjacent box beams.

C14.7.6.3.6

Delete AC14.7.6.3.6.

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where:

- L = length of a rectangular bearing (in.)
 W = width of a rectangular bearing (in.)
 D = diameter of circular bearing (in.)
 H = total height of bearing including thickness of shims
 in reinforced bearing (in.)

*14.7.6.3.7 Reinforcement**C14.7.6.3.7*

The following shall replace A14.7.6.3.7.
 Reinforcement for steel-reinforced elastomeric bearings designed in accordance with the provisions of this article shall conform to the requirements of D14.7.5.3.5.

Delete AC14.7.6.3.7.

14.7.6.3.8 Seismic and Other Extreme Event Provisions

The following shall replace the last sentence of A14.7.6.3.8.

The provisions of A14.7.5.3.7 and D14.7.5.3.7 shall also apply as applicable.

*14.7.6.3.9P Bearing Area**14.7.6.3.9aP General**C14.7.6.3.9aP*

Epoxy-coated bearing surfaces are not to be specified.

The contact surfaces above and below neoprene bearing pads shall be roughened to prevent excessive pad bulging and walking out. Sandblast clean the concrete bearing surface to achieve a rough texture.

The Department has experienced instantaneous failure of bearing pads when the bearing surfaces were epoxy-coated. To avoid the high costs of corrective measures, the Department no longer permits epoxy-coated bearing surfaces.

14.7.6.3.9bP Prestressed Adjacent Box Beams

See D14.7.6.3.9dP for bearing area and beveled sole plate requirements.

14.7.6.3.9cP Prestressed Spread Box Beams and Prestressed I-Beams

See D14.7.6.3.9dP for bearing area and beveled sole plate requirements.

14.7.6.3.9dP Beam Seat and Bottom of Beam Bearing Area Requirements

Bearing pads require uniform pressure, insofar as practical, over the entire bearing area of the pads to assure long-term, maintenance-free performance. Therefore, it is essential to ensure that the planes formed by the bottom of the beam bearing area and the top of beam seat are parallel unless the bearing pad is specifically designed to accommodate the slope differential. This requirement applies to both the longitudinal and transverse axes of the bearing pad. Refer to Figure 14.7.6.3.9dP-1 for beam and bearing axes orientation.

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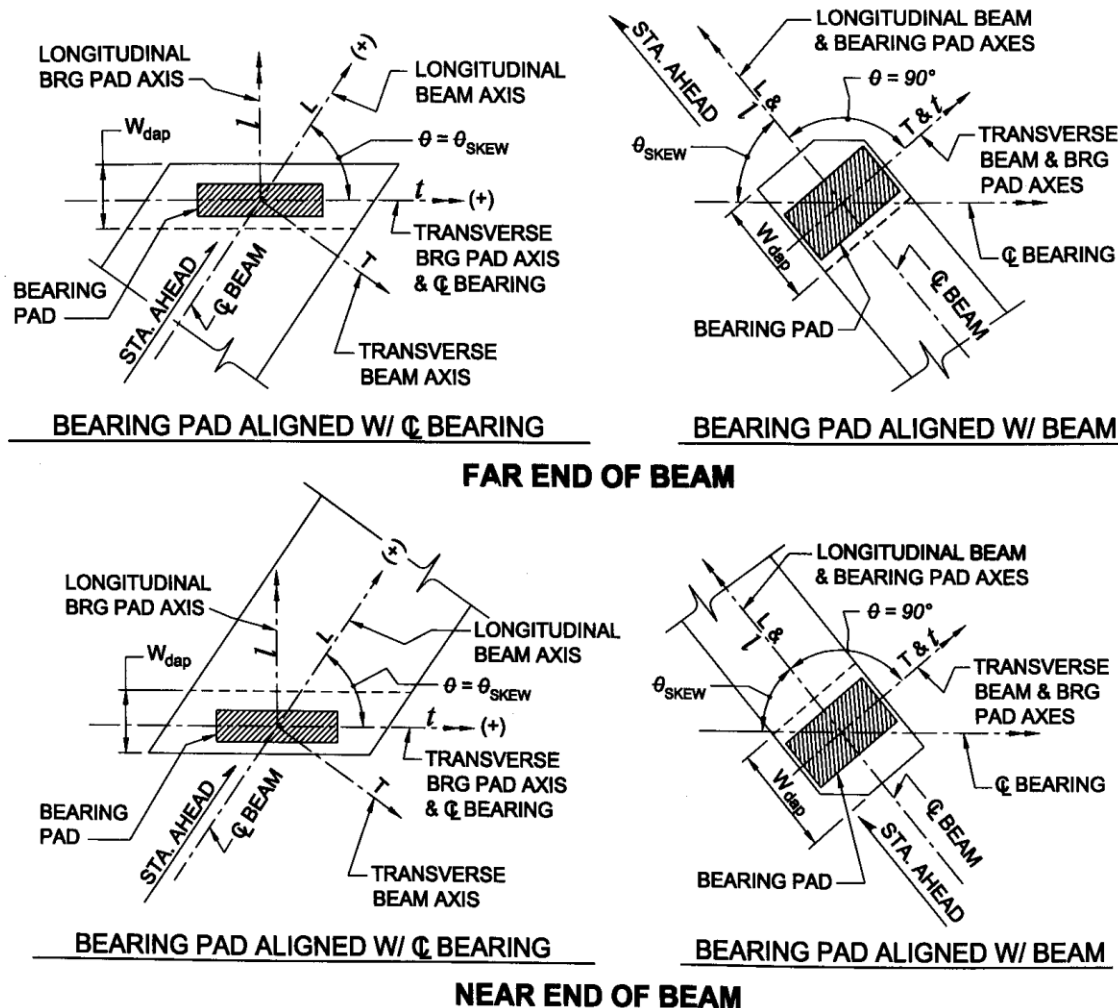


Figure 14.7.6.3.9dP-1 - Beam & Bearing Pad Axes Orientation

14.7.6.3.9d.1P Design of Beam Seat

14.7.6.3.9d.1.1P Direction Parallel to Longitudinal Axis of Beam

The beam seat in the direction parallel to the longitudinal axis of the beam may be constructed to a maximum slope of 1%. Any remaining differential slope between the beam seat and the bottom of beam bearing area must be accommodated for in the design of the bearing pad or by modifying the bottom of beam bearing area in accordance with D14.7.6.3.9d.2.1P.

14.7.6.3.9d.1.2P Direction Parallel to Transverse Axis of Bearing Pad

Establish the beam seat elevations/slopes in the direction parallel to the transverse axis of the bearing pad so that the beam seat is parallel to the bottom of beam. Doing so will ensure parallel top and bottom planes for the bearing

C14.7.6.3.9d.1.2P

Past experience has shown that the beams have a tendency to slide transversely and to distort the bearing pad if not temporarily braced.

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pad in this direction. Furthermore, if modification of the bottom of beam bearing area is necessary, properly sloping the beam seat in the direction parallel to the transverse axis of the bearing pad will eliminate the need for two-dimensional sloping of the bottom of beam bearing area. For beams set truly vertical, such as I-beams, the beam seat will be level (see D14.7.6.3.9d.2.3P). There is no maximum slope to which the beam seat may be constructed in this direction. However, for box beams having a transverse beam seat slope, s_t , exceeding 5% and placed on neoprene bearing pads thicker than 3.5 in., providing a note on the design drawings requiring the contractor to provide temporary lateral support to the beam during construction until the end diaphragms are cast and the shear blocks or dowel bars are installed.

14.7.6.3.9d.2P Design of Bottom of Beam Bearing Area

14.7.6.3.9d.2.1P Direction Parallel to Longitudinal Axis of Beam

When the slope of the bottom of beam in the direction parallel to the longitudinal axis of the beam, S_L , cannot be accommodated for in the design of the bearing pad in conjunction with a maximum beam seat slope of 1%, modification of the bottom of beam bearing area is required. Acceptable methods for modifying the bottom of beam bearing area are required. Acceptable methods for modifying the bottom of beam bearing area include:

- For steel beams, provide a beveled steel sole plate;
- For all prestressed concrete beams, cast a beveled notch in the underside of the beam (referred to as dapping) unless the slope exceeds the maximum permitted slope, $(S_L)_{\max}$ (see below), in which case provide a beveled steel sole plate. The slope of the beam dap/beveled steel sole plate is to match the actual bottom of the beam slope in the direction parallel to the longitudinal axis of the beam computed in accordance with D14.7.6.3.9d.2.4P.

The maximum slope in the direction parallel to the longitudinal axis of the beam that can be accommodated by dapping is variable and can be expressed as:

$$(S_L)_{\max} = [(t_{\text{dap}})_{\max} - (t_{\text{dap}})_{\min}] \times \sin \theta / W_{\text{dap}} \quad (14.7.6.3.9d.2.1P-1)$$

where,

$(t_{\text{dap}})_{\max} =$ 1.5 in. based on a maximum distance from the bottom of the beam to the centerline of the prestressing strand bottom row of 3.25 in. and minimum concrete covers of 1.5 in. on 1/2 in. diameter prestressing strands and 1 in. on #4 confinement reinforcement bars in the beam

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dapping area. Do not increase the maximum dap thickness beyond 1.5 in. regardless of other methods used to increase concrete cover on the confinement reinforcement (such as using draped strand design.)

$(t_{dap})_{min} = 0.25$ in. based on input from prestressed concrete beam fabrication industry regarding minimum practical thickness that shims used to form daps can be constructed.

$W_{dap} =$ Maximum width of beam dap measured parallel to longitudinal axis of bearing pad.

$= d_{brg} + L_{bp}/2 + d_{clr}$ (Refer to Figures D14.7.6.3.9dP-1 & D14.7.6.3.9d.2.4P-1)

$d_{brg} =$ distance from end of beam to C/L bearing measured parallel to longitudinal axis of bearing pad.

$L_{bp} =$ length of bearing pad measured parallel to longitudinal axis of bearing pad.

$d_{clr} =$ distance from edge of bearing pad to end of beam dap measured parallel to longitudinal axis of bearing pad. The minimum value for $d_{clr} = 3$ in. to account for construction tolerances of beam length and centerline of bearing location.

$\theta =$ Counterclockwise angle formed from the positive (+) transverse bearing pad axis to the positive (+) longitudinal beam axis (refer to Fig. D14.7.6.3.9dP-1).

Partial-width beam daps (see Figure 14.7.6.3.9d.2.1P-1) are **not** permitted. The formed notched area for the dap must extend completely to the end of the beam.

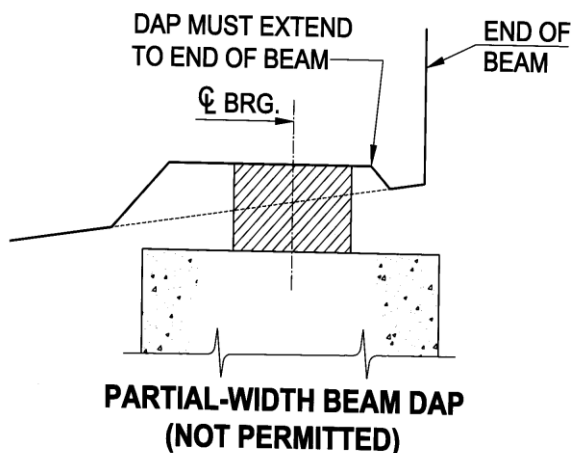


Figure 14.7.6.3.9d.2.1P-1 – Partial-Width Beam Dap

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14.7.6.3.9d.2.2P Direction Parallel to Transverse Axis of Bearing Pad

Since the beam seat is to be constructed parallel to the actual slope of the bottom of the beam (see D14.7.6.3.9d.1.2P), there are no special construction requirements for the bottom of the beam bearing area in the direction parallel to the transverse axis of the bearing pad.

14.7.6.3.9d.2.3P Transverse Slope of Beams Relative to Beam Axis

Set the transverse beam slope, S_T , as follows (refer to Figure 14.7.6.3.9d.2.3P-1):

Steel and Prestressed

Concrete I-beams Set beams truly vertical in all cases.

Spread Box Beams Set beams truly vertical or on a slope to conform to the deck cross-slope. When setting on slope, special considerations are required in areas of superelevation transition or within a vertical curve profile with skewed supports. When setting beams vertical, properly consider effects of haunch thickness on beam design and detailing, specifically, the additional weight of concrete and the need for haunch reinforcement.

Adjacent Box/Plank
Beams

Set beams to conform as closely as practical to the deck cross-slope in order to minimize the haunch thickness and to align holes for the transverse post-tensioning tendons. In areas of superelevation transition or within a vertical curve profile with skewed supports, additional haunch or stepped beam seats may be required.

For box beams, ensure that the transverse beam slope is the same at each end of the beam to prevent inducing torsion.

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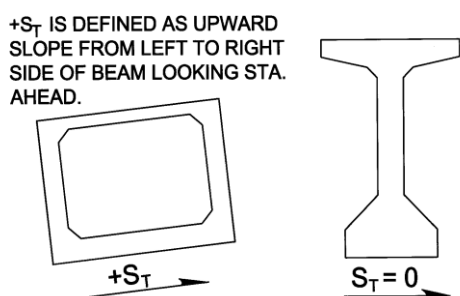


Figure 14.7.6.3.9d.2.3P-1 – Transverse Beam Slope

14.7.6.3.9d.2.4P Computation of Beam Dap Dimensions

Establish the beam dap dimensions to achieve a level bottom of beam bearing area in the direction parallel to the longitudinal axis of the beam. Compute the required beam dap thicknesses as follows (refer to Figure 14.7.6.3.9d.2.4P-1):

If S_L is positive (i.e., sloping upward in the stations ahead direction):

$$(t_{dap})_B = (t_{dap})_{min} + S_L \times W_{dap} / \sin \theta \quad (14.7.6.3.9d.2.4P-1)$$

$$(t_{dap})_A = (t_{dap})_{min} \quad (14.7.6.3.9d.2.4P-2)$$

If S_L is negative (i.e., sloping downward in the stations ahead direction):

$$(t_{dap})_B = (t_{dap})_{min} \quad (14.7.6.3.9d.2.4P-3)$$

$$(t_{dap})_A = (t_{dap})_{min} - S_L \times W_{dap} / \sin \theta \quad (14.7.6.3.9d.2.4P-4)$$

where:

$(t_{dap})_B$ = beam dap thickness at the back station end of the notch;

$(t_{dap})_A$ = beam dap thickness at the ahead station end of the notch;

S_L = bottom of beam slope at the end of the beam under consideration in the longitudinal beam axis direction,
= $(S_L)_G + (S_L)_C$

$(S_L)_G$ = longitudinal slope due to grade of beam,
= $[(E_b)_f - (E_b)_n] / L$

$(E_b)_n$ = Bot. of beam elev. at C/L brg. at near end (ft.)

$(E_b)_f$ = Bot. of beam elev. at C/L brg. at far end (ft.)

L = beam span length, c/c bearing (ft.)

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$(S_L)_C$ = longitudinal slope due to beam camber.
 $(S_L)_C$ is a positive (+) value at the near end of the beam and a negative (-) value at the far end of the beam.
 $= 4 y_c/L$

y_c = beam net final camber at mid-span (ft.)

Round dap thicknesses to the nearest 1/16 in..

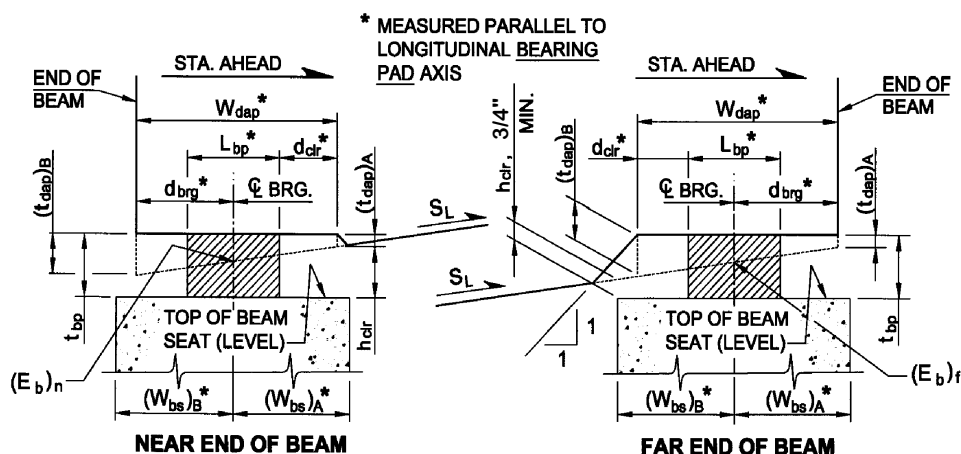


Figure 14.7.6.3.9d.2.4P-1 - Typical P/S Concrete Beam Dap Detail (Viewed Parallel to Longitudinal Beam Axis)

14.7.6.3.9d.2.5P Computation of Beam Seat Elevations when Providing Beam Dap

Establish the beam seat elevations/slope that the beam seat in the transverse direction of the bearing pad is parallel to the bottom of beam bearing area slope. Compute the beam seat elevations as follows (refer to Figure 14.7.6.3.9d.2.5P-1):

- 1) Compute beam dap thickness at C/L bearing:

For the **near end** of beam (back station):

$$(t_{dap})_C = (t_{dap})_B + [(t_{dap})_A - (t_{dap})_B] \times d_{brg} / W_{dap} \quad (14.7.6.3.9d.2.5P-1)$$

For the **far end** of beam (ahead station):

$$(t_{dap})_C = (t_{dap})_A + [(t_{dap})_B - (t_{dap})_A] \times d_{brg} / W_{dap} \quad (14.7.6.3.9d.2.5P-2)$$

- 2) Beam seat elevation at C/L beam along C/L bearing:

$$(E_{bs})_C = (E_b)_{n \text{ or } f} + [(t_{dap})_C]_{n \text{ or } f} - (t_{bp})_{n \text{ or } f} \quad (14.7.6.3.9d.2.5P-3)$$

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where:

$(t_{bp})_{n \text{ or } f}$ = thickness of bearing pad at the near/far end of beam.

3) Beam seat elevations at the four corners of the beam seat can be computed as follows:

$$(E_{bs})_{BR} = (E_{bs})_c + s_t (L_{bs}/2) - s_1 (W_{bs})_B \quad (14.7.6.3.9d.2.5P-4)$$

$$(E_{bs})_{BL} = (E_{bs})_c - s_t (L_{bs}/2) - s_1 (W_{bs})_B \quad (14.7.6.3.9d.2.5P-5)$$

$$(E_{bs})_{AR} = (E_{bs})_c + s_t (L_{bs}/2) + s_1 (W_{bs})_A \quad (14.7.6.3.9d.2.5P-6)$$

$$(E_{bs})_{AL} = (E_{bs})_c - s_t (L_{bs}/2) + s_1 (W_{bs})_A \quad (14.7.6.3.9d.2.5P-7)$$

where:

L_{bs} = Length of beam seat assumed centered about C/L beam and assumed rectangular in shape (plan view).

$(E_{bs})_{BR}$ = Beam seat elevation at right, back station corner.

$(E_{bs})_{BL}$ = Beam seat elevation at left, back station corner.

$(E_{bs})_{AR}$ = Beam seat elevation at right, ahead station corner.

$(E_{bs})_{AL}$ = Beam seat elevation at left, ahead station corner.

$(W_{bs})_A$ = Width of beam seat from C/L bearing to ahead station face/edge.

$(W_{bs})_B$ = Width of beam seat from C/L bearing to back station face/edge.

s_t = Bottom of beam slope in the transverse bearing pad axis direction,

$$= S_L \cos \theta + S_T \sin \theta$$

S_T = Bottom of beam slope in transverse beam axis direction,

= 0 for beams set vertically (steel and prestressed concrete I-beams), Value is positive (+) if beam slopes upward from left-to-right looking stations ahead.

s_1 = Bottom of beam slope within beam dap area in the longitudinal bearing pad axis direction,
= $-s_t / \tan \theta$

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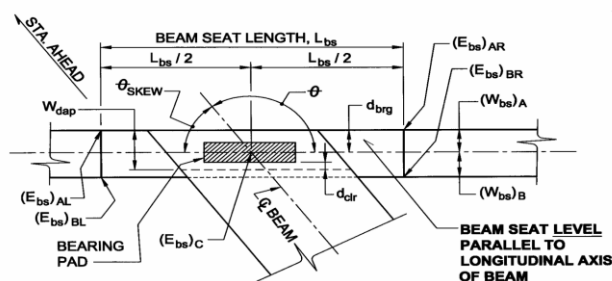


Figure 14.7.6.3.9d.2.5P-1 – Typical Beam Seat Detail (Far End of Beam Shown)

Note: For the case in which the bearing pads are aligned relative to the beam and not the C/L bearing (typically, steel & prestressed I-beams), the beam seat is to be constructed LEVEL in both the longitudinal and transverse directions. Therefore, only the beam seat elevation at C/L beam must be computed.

14.7.6.3.9d.2.6P Additional Box Beams Requirements When Dapping

When dapping is used on spread and adjacent box beams, ensure that a minimum concrete cover of 1.25 in. is provided on the prestressing strands adjacent to the inner void. Increase the thickness of the bottom flange in 1/2 in. minimum increments to meet this requirement.

14.7.6.3.9d.2.7P Minimum Clearance for Inspectability

Provide 3/4 in. minimum clearance from the underside of beam (without consideration of the beam dap) to the top of beam seat to ensure inspectability of the pad. Increase the thickness of the bearing pad in order to satisfy this requirement.

The actual clearance provided, h_{clr} , can be computed from the following equations:

For the **near end** of beam (back station):

$$\text{If } S_L \text{ is positive,} \\ h_{clr} = t_{bp} - (t_{dap})_A \quad (14.7.6.3.9d.2.7P-1)$$

$$\text{If } S_L \text{ is negative,} \\ h_{clr} = t_{bp} - (t_{dap})_C + (W_{bs})_A \times S_L / |\sin \theta_{SKEW}| \quad (14.7.6.3.9d.2.7P-2)$$

For the **far end** of beam (ahead station):

$$\text{If } S_L \text{ is positive,} \\ h_{clr} = t_{bp} - (t_{dap})_C - (W_{bs})_B \times S_L / |\sin \theta_{SKEW}| \quad (14.7.6.3.9d.2.7P-3)$$

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If SL is negative,

$$h_{clr} = t_{bp} - (t_{dap})_B \quad (14.7.6.3.9d.2.7P-4)$$

14.7.6.3.10P Shear Strain Components for Isolation Design

C14.7.6.3.10P

The various components of shear strain in the bearing are computed as follows:

The shear strain, ε_{sc} , due to compression by vertical loads is directly dependent on the compression strain, ε_c , due to the vertical loads. In computing ε_c , an apparent compressive modulus is used such that

- (a) Shear strain, ε_{sc} , due to compression by vertical loads is given by

$$\varepsilon_{sc} = 6 S \varepsilon_c \quad (14.7.6.3.10P-1)$$

$$\varepsilon_c = \frac{P}{A_r E_c} \quad (C14.7.6.3.10P-1)$$

For $S \geq 3$

For $S \geq 3$

$$\varepsilon_c = \frac{\Delta_{ci}}{\sum h_{ri}} = \frac{\Delta_c}{h_{rt}} = \left[\frac{P}{A_r (5GS^2)} \right] \quad (14.7.6.3.10P-2)$$

$$E_c = 5GS^2 \quad (C14.7.6.3.10P-2)$$

in which the elastomer shear modulus, G , depends on the specific bearing material with a range of 0.1-0.2 ksi. For the rare case in which $S < 3$,

For $S < 3$

$$\varepsilon_c = \frac{\Delta_{ci}}{\sum h_{ri}} = \frac{\Delta_c}{h_{rt}} = \left[\frac{P}{A_r E_c (1 + 2k S^2)} \right] \quad (14.7.6.3.10P-3)$$

$$E_c = E(1 + 2kS^2) \quad (C14.7.6.3.10P-3)$$

in which E_c is the modulus of elasticity of the elastomer and k is a material property. The British Specification BE 1/76 (1) provides guidance for the selection of appropriate values for E and k .

where:

$$A_r = A_b (1 - \Delta/B)$$

For most practical cases $S \geq 3$

The effects of creep of the elastomer shall be added to the instantaneous compressive deflection, Δ_c , when considering long-term deflections. For compressive deflection requirements see D14.7.6.3.3. They shall be computed from information relevant to the elastomer compound used, if it is available. If not, the material properties given in A14.7.5.2 shall be used as a guide.

- (b) Shear strain, ε_{sh} , due to imposed lateral displacement is given by

$$\varepsilon_{sh} = \frac{\Delta_s}{h_{rt}} \quad (14.7.6.3.10P-4)$$

- (c) Shear strain, ε_{eq} , due to earthquake-imposed displacement is given by

$$\varepsilon_{eq} = \frac{d_i}{h_{rt}} \quad (14.7.6.3.10P-5)$$

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(d) Shear strain, ε_{sr} , due to rotation is given by

$$\varepsilon_{sr} = \frac{B^2 \theta}{(2h_{ri}h_{rt})} \quad (14.7.6.3.10P-6)$$

14.7.6.3.11P Limiting Criteria for Allowable Vertical Loads

The allowable vertical load on an elastomeric isolation bearing is not specified explicitly. The limits on vertical load are governed indirectly by limitations on the equivalent shear strain in the rubber due to different load combinations and to stability requirements. The permissible shear strain in the rubber is expressed as ϕ times the minimum specified elongation-at-break (ε_u). The value of ϕ is dependent on the load combination under consideration with specific values given in the following sections.

14.7.6.3.11aP Service Load Combinations

The following criteria shall be satisfied for service loads, which include dead load plus live load, thermal, creep, shrinkage and rotation:

$$(a) \quad 0.5 \varepsilon_u \geq \varepsilon_{sc} + \varepsilon_{sh} + \varepsilon_{sr} \quad (14.7.6.3.11aP-1)$$

$$(b) \quad 0.33 \varepsilon_u \geq \varepsilon_{sc} \quad (14.7.6.3.11aP-2)$$

The smaller load factors for TU, CR and SH should be used per D3.4.1.

14.7.6.3.11bP Seismic Load Combinations

The following criterion shall be satisfied for seismic loads which include dead load and seismic live load, seismic design displacements and rotation:

$$0.75 \varepsilon_u \geq \varepsilon_{sc} + \varepsilon_{eq} + \varepsilon_{sr} \quad (14.7.6.3.11bP-1)$$

If thermal displacements are large, ε_{sh} shall be included on the right-hand side of the above inequality.

C14.7.6.3.11P

Since the primary design parameter for earthquake loading is the displacement, d_i , of the bearing, the design procedures shall be capable of incorporating this displacement in a logical and consistent manner. The requirements of D14.7.6.3 limit vertical loads by using a limiting compressive stress. Therefore, they do not have a mechanism for including the simultaneous effects of seismic displacements. The British Specification BE 1/76 (1976), and its more recent successor BS 5400 (1981), recognize that shear strains are induced in reinforced bearings by compression, shear deformation, and rotation. In these codes, the sum of these shear strains is limited to a proportion of the elongation-at-break of the rubber. The proportion (1/2 or 1/3 for service load combinations and 3/4 for seismic load combinations) is a function of the loading type.

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14.7.6.3.11cP Stability Against Overturning

Elastomeric isolation bearings shall be shown either by test or analysis to be capable of resisting the dead load, seismic live load, and any vertical load resulting from overturning at 1.5 times the seismic design displacement, d_i .

14.7.6.4P Anchorage

If the factored shear force sustained by the deformed pad at the service limit state exceeds one-fifth of the compressive force, P_{sd} , due to permanent loads, the pad shall be secured against horizontal movement.

When the effects of live load are included in the factored shear force in the above requirements, the corresponding reaction from the live load may be included in the compressive force when checking the one-fifth limitation.

For spans less than 50 ft., when the above requirements are not satisfied, holes for dowels in the bearings will be permitted to secure pads against displacement. The effect of holes must be accounted for in the design.

Securing pads against displacement by the use of adhesive is not permitted.

These provisions also apply to steel-reinforced elastomeric bearings designed using Method B.

14.7.6.5P Drawing Requirements

The contract drawings shall provide the following information:

1. Bearing pad length, width, thickness, edge cover, and layer thickness.
2. Total number of pads required for the project and number required for the testing.
3. Number of shims and shim thickness.
4. Project Specific Notes.
5. As applicable, sole plate sizes, material, finish, and type of coating.
6. Bearing seat or sole plate slope.
7. As applicable, anchor bolt sizes, lengths, and material requirements.
8. Reference to BC-755M

14.7.7 Bronze or Copper Alloy Sliding Surfaces**14.7.7.1 Materials**

The following shall supplement A14.7.7.1.
For bronze/carbon steel sliding surfaces, the surface finish shall not be more than 125 μ -in. rms.

C14.7.6.4P

The DM-4 is using service loads for anchorage check because the temperature range for the design of bearings in the DM-4 takes into account possible variations in setting temperature.

When checking the one-fifth anchorage limitation with live load effects, the live load reaction may be computed as:

Simple Spans: One-half the total live load force effect as specified in D2.5.2.6.2.

Continuous Spans and Simple Spans made Continuous: The determination of live load reaction concomitant with longitudinal movement in the bearing due to live load is a difficult task. As such, downward live load reaction should normally be neglected when checking anchorage requirements for continuous spans. Uplift reactions should be considered.

C14.7.6.5P

The inclusion of this information on the contract drawings in conjunction with material and fabrication tolerance requirements specified in Publication 408 eliminate the need for shop drawings for elastomeric bearing pads.

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14.7.7.5P Rotational Elements - Concave - Spherical

The spherical radius shall be determined such that the resulting geometry of the bearing is capable of withstanding the greatest ratio of horizontal force to vertical load under all loading conditions to prevent unseating the concave element.

If required, mechanical safety restraints shall be considered to prevent overturning.

Maximum design rotation of the structure, which includes a 0.02 radian construction tolerance, shall be considered in the bearing design to prevent overturning or uplift.

Calculations showing the determination of the radius shall be submitted for approval.

The bearing surfaces shall have lubricant recesses consisting of concentric rings with or without central circular recesses with a depth at least equal to the width of the rings or recesses.

The recesses or rings shall be arranged in a geometric pattern so that adjacent rows overlap in the direction of motion.

The entire area of all bearing surfaces that have provision for relative motion shall be lubricated by means of the lubricant-filled recesses.

The lubricant-filled areas shall comprise not less than 25% of the total bearing surface.

The lubricant compound shall be integrally molded at high pressure and compressed into the rings or recesses and project no less than 0.01 in. above the surrounding bronze plate.

14.7.8 Disc Bearings**14.7.8.1 General**

The following shall supplement A14.7.8.1.

PADOT DWG #97-601-BQAD Standard Disc Bearing Details shall be used for disc bearings with vertical loads between 100 kips and 1000 kips with 0.02 radians maximum rotation. If the standard is not used, complete design calculations must be provided prior to submission of shop drawings on all projects for all aspects of disc bearings.

For high load multi-rotational bearing design plan presentation, see D14.6.1.2.1P.

14.7.8.3 Elastomeric Disc

The following shall supplement A14.7.8.3.

The area of a disc shall be designed for a maximum nominal resistance of 3.7 ksi for Polyether Urethane Compound A and 5 ksi for Polyether Urethane Compound B at the total dead and live loads of the structure.

14.7.8.4 Shear Resisting Mechanism**C14.7.8.4**

The following shall replace the first paragraph of

Delete the third paragraph of AC14.7.8.4.

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A14.7.8.4

In fixed and guided bearings, a shear-resisting mechanism shall be provided to transmit horizontal forces between the upper and lower steel plates. It shall be capable of resisting a horizontal force in any direction equal to the larger of the design shear force at service or extreme event I limit states or 10% of the maximum vertical load at the service limit state at that location.

14.7.8.5 Steel Plates

The following shall replace the second paragraph of A14.7.8.5.

The section thickness of the plate beneath the disc where not limited by bending stresses shall not be less than:

- Disc OD x 0.06 for bearing directly on concrete, but not less than 3/4 in.
- Disc OD x 0.045 for bearing directly on steel masonry plates, but not less than 1/2 in.

The following shall supplement A14.7.8.5.

A limiting ring shall be provided by a welded ring or by machining a recess in the bearing plate. The depth of the limiting ring shall be equal to or greater than ID x 0.014. The inside diameter of the retainer ring shall be greater than the diameter of the disc by 4% to 6% of the diameter.

Bearing plates shall be connected to masonry plates by means of a fillet weld around the entire perimeter. Full clearance shall be maintained between the bearing parts for the following condition:

Design Rotation (radians) + vertical deflection

Refer to Figure 14.7.8.5-1P for disc bearings.

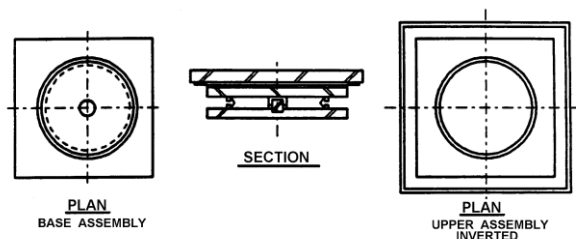


Figure 14.7.8.5-1P - Disc Bearings

14.7.9 Guides and Restraints

The following shall supplement A14.7.9.

Guided members shall have their contact area within the guide bars in all operating positions. Guiding off the fixed base or any extensions of it is not permitted. Do not use inner key to resist horizontal forces. Instead, use guide bars.

Alignment of bearing guiding systems relative to the anticipated movement direction of the structure shall be carefully considered to avoid bearing guide system failure.

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Special studies or designs may be required on curved or skewed structured to ensure correct alignment.

14.7.9.1 General

The following shall supplement A14.7.9.1.

Guide bars may be integral by machining from the solid or by welding or connecting with high-strength fasteners. High-strength fasteners shall be designed using $0.25 \times$ Ultimate Shear Strength.

PTFE used on guide bars shall be pigmented.

14.7.9.2 Design Loads**C14.7.9.2**

The following shall replace A14.7.9.2.

Guides or restraints shall be designed at the service limit state for the larger of either:

- The horizontal force at the service limit state, or
- 10 percent of the maximum vertical load at the service limit state at that location

Delete the third paragraph of AC14.7.9.2

Guides and restraints shall be designed for applicable seismic forces using the extreme event I limit state and the provisions of A3.10.9 and D3.10.9.

14.7.9.4 Geometric Requirements

The following shall supplement A14.7.9.4.

Total spacing should not be specified more than 1/16 in. between guides and guided components where possible (see A14.7.9).

Clearance between bearing guides or keys and guided members shall be 1/16 in..

14.7.9.6 Attachment of Low-Friction Material

The following shall supplement A14.7.9.6.

PTFE must be used on guides and shall be bonded to and recessed in their substrate. The PTFE shall also be mechanically fastened by a minimum of two screws. The screws shall be recessed a minimum of 50% of the amount of protrusion of the PTFE above the guiding surface.

14.8 LOAD PLATES AND ANCHORAGE FOR BEARINGS**14.8.1 Plates for Load Distribution**

The following shall supplement A14.8.1.

The stainless steel plate mounted on sole plates to permit full expansion and contraction for pot or disc bearings should be extended to within 1/4 in. of the edge of the sole plate. This will minimize a need for costly field adjustment in case of design or construction error.

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14.8.2 Tapered Plates

The following shall supplement A14.8.2.

Sole plate above the bearings shall be tapered as needed so that the distance between the top of the pot and the sliding surface does not deviate from general uniformity by more than 1/8 in. under all dead loads.

14.8.3 Anchorage and Anchor Bolts**14.8.3.1 General**

The following shall replace the third paragraph of A14.8.3.1.

Trusses, girders and rolled beams shall be securely anchored to the substructure. Preferably, anchor bolts shall be cast in substructure concrete; otherwise, they may be grouted in place. Anchor bolts may be swaged or threaded to secure a satisfactory grip upon the material to embed them in the holes. Chief Bridge Engineer approval is required for the use of swaged anchor bolts for cases when the anchor bolt will experience direct tension. Dowels shall be cast in the substructure concrete.

The following shall supplement A14.8.3.1.

The factored shear resistance of anchor bolts and dowels shall be greater than the factored tensile horizontal or transverse forces due to the strength limit states and all applicable extreme event load combinations.

The embedment length of the dowel or anchor bolt in shear may need to be further increased due to the bearing resistance of the concrete.

Anchor bolts detailed in accordance with BC-755M, resulting in unsupported anchor bolt length, shall be analyzed for bending stress. Anchor bolt details utilizing a double nut and leveling nut connection (i.e. sign structures), where the clearance between the top of concrete foundation and bottom of leveling nut exceeds D_b , shall consider bending stress in the anchor bolt. The factored bending resistance of the anchor bolts shall be greater than the factored bending stress due to the strength limit states and all applicable extreme event load combinations. In determining the factored forces on anchor bolts at the strength and extreme event limit states, anchor bolts from multiple elastomeric bearings are assumed to be engaged, and resist the factored transverse forces. The factored bending resistance of anchor bolts shall be calculated as specified in Eq. A6.12.2.2.7-1. If applicable, the presence of threads should be considered in determining the resistance.

14.8.3.3P Anchor Bolts

For each bearing, Table 14.8.3.3P-1 provides minimum anchor bolt requirements.

Detail with a double nut above the base plate and a leveling nut below the base plate, when properly tensioned, results in the anchor bolt behaving as a fixed-fixed beam due to horizontal loads. Thus, for clearances between leveling nut and top of concrete less than D_b , bending does not need to be considered.

SPECIFICATIONS

COMMENTARY

Table 14.8.3.3P-1 - Minimum Anchor Bolt Requirements

Type	Span Range (L) (ft.)	No. of Bolts	Diameter of Bolts (in.)	Minimum Embedment Length (in.)
Rolled Beams	All	2	1	10
Girders and Trusses	$L \leq 50$	2	1	10
Girders and Trusses	$50 < L \leq 100$	2	1 1/4	12
Girders and Trusses	$100 < L \leq 150$	2	1 1/2	15
Girders and Trusses	$150 < L$	4	1 1/2	15

14.8.3.4P Dowels

For prestressed concrete girders when anchorage bolts are not used, dowels can be used to prevent longitudinal and transverse movements. Standard Drawings BD-656M, BD-664M and BD-665M provide minimum requirements for dowels.

14.10 REFERENCES

The following shall supplement A14.10.

British Standards Institution, Draft British Standard BD 5400: Steel, Concrete and Composite Bridges: Part 9A: Code of Practice for Design of Bearings, and Part B: Specifications for Materials, Manufacture and Installation of Bearings, Document 81/10/84, 1981

"Design Requirements for Elastomeric Bridge Bearings", Technical Memorandum BE 1/76, Highways Directorate, Department of Environment, Great Britain, 1976

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX A i QUALITY ASSURANCE FORMS FOR BRIDGE DESIGN

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1.0 GENERAL

The intent of these forms is to self-correct critical omissions and errors during the design of bridge components. All applicable forms will be filled out by the actual bridge designer, either in-house or consultant, and will be included with the appropriate submission, as given below, to the reviewing office, either District or Central Office. It is anticipated that it will require approximately 15 minutes to complete each form, and as such, should not pose any undue or additional burden on the designer. Thus, these forms shall be completed at no additional cost to the Department.

Form D-500, Engineering Agreements, must be completed for each structure and included as a part of the technical proposal for an Engineering Agreement or as a part of the Work Order under an Open-End Contract. This form should be incorporated in an appendix along with the District's Scope of Work.

Form D-501 through D-504 must be completed, as appropriate, and included as a part of the TS&L submission.

Form D-505, Foundations, must be completed and included as part of the Foundation submission.

Forms D-506 through D-518 must be completed, as appropriate, and included as part of the Final Design submission. Form D-513, BRADD Final Plans, must be completed and included with all BRADD designed projects, either in-house or consultant, along with all the other pertinent forms required with the Final Design submission.

Any submission received by the approving authority, either District or Central Office, without all the completed applicable forms attached will be returned without any action taken.

These forms also apply to all submissions developed by the contractor for alternate designs in accordance with PP1.10.1.

Form D-519 is intended for Department Use Only for reviews of Construction Load Analysis submissions performed by Central Office.

2.0 QUALITY ASSURANCE FORMS

A copy of each form is given on the following pages of this appendix.

D-500 (5-12)



QUALITY ASSURANCE FORM FOR ENGINEERING AGREEMENTS

Note: Provide one form for each structure included under this technical proposal or work order

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Engineering Agreement No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

Structure Identification: _____

2. PROJECT TYPE

☐ Rehabilitation ☐ Replacement ☐ Bridge at New Location

3. PROJECT STAGE

☐ Preliminary Design ☐ Final Design ☐ Construction

☐ Other/Specify: _____

4. PRELIMINARY DESIGN (Key Items Only)

(Check Applicable Box)

A. Hydraulics

	Yes	No	N/A
Is Backwater Study Addressed?	☐	☐	☐

B. New Bridge

Alternate Location and Bridge Type Study:			
Preliminary Geotechnical Investigation Required?.....	☐	☐	☐

Other: _____	☐	☐	☐

Comments: _____

C. Bridge Rehabilitation

Bridge Type and Spans: _____

Are the Following Items Included?	Yes	No	N/A

Current Inspection Report Study to be Used	☐	☐	☐

Detailed Inspection Report Part of Agreement	☐	☐	☐

Fatigue Evaluation	☐	☐	☐

Scour Evaluation	☐	☐	☐

Seismic Evaluation	☐	☐	☐

D-500 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

	(Check Applicable Box)		
	Yes	No	N/A
Preliminary Rehabilitation Alternate Study	☐	☐	☐
Fracture Criticality	☐	☐	☐
Bridge Paint Evaluation.....	☐	☐	☐
Other: _____	☐	☐	☐
Comments: _____			

5. FINAL DESIGN

Will BRADD be Used?	☐	☐	☐
BRADD Input will be Prepared and Output Reviewed?	☐	☐	☐

A. T.S.&L. Stage

Will the Following Key Items be Studied?

Fatigue.....	☐	☐	☐
Scour	☐	☐	☐
Seismic	☐	☐	☐
Bearings	☐	☐	☐
Load Capacity Study	☐	☐	☐
Drainage (Superstructure & Substructure)	☐	☐	☐
Deck Type	☐	☐	☐
Elimination of Expansion Joints	☐	☐	☐
Structural Stability/Constructability	☐	☐	☐
Structure Inspectability and Maintainability	☐	☐	☐
Approach Slab and Pavement Relief Joints	☐	☐	☐
Maintenance & Protection of Traffic	☐	☐	☐
Construction Sequencing of All Operations	☐	☐	☐

D-500 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

(Check Applicable Box)

Yes No N/A

Analysis Method Used: ☐ PennDOT Approx. ☐ 2D ☐ 3D

Number of Alternates to be Studied: ☐ Three ☐ Four or More

Number of Spans (Existing) or (Anticipated) _____

Other: _____ ☐ ☐ ☐

Comments: _____

B. Foundation

Geotechnical Investigation ☐ ☐ ☐

Foundation Design ☐ ☐ ☐

Scour ☐ ☐ ☐

Other: _____ ☐ ☐ ☐

Comments: _____

C. Final Plans

Will the Following Key Items be Addressed, Designed and Detailed?

Fatigue ☐ ☐ ☐

Seismic ☐ ☐ ☐

Bearings ☐ ☐ ☐

Deck Design ☐ ☐ ☐

Beam Design ☐ ☐ ☐

Substructure Design ☐ ☐ ☐

Drainage ☐ ☐ ☐

Continuity Design ☐ ☐ ☐

Jacking of Superstructure ☐ ☐ ☐

D-500 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

	(Check Applicable Box)		
	Yes	No	N/A
Concrete and Crack Repair	☐	☐	☐
Structural Stability/Constructability	☐	☐	☐
Structure Inspectability and Maintainability	☐	☐	☐
Approach Slab and Pavement Relief Joints	☐	☐	☐
Maintenance & Protection of Traffic	☐	☐	☐
Construction Sequencing of All Operations	☐	☐	☐
Other: _____	☐	☐	☐
Comments: _____			

NOTE: These are not all-inclusive lists. Refer to applicable Design Manuals or additional items.

D-500 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Engineering Agreement No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
 (STREAM, RAILROAD, OR ROAD)

Structure Identification: _____

6. ENGINEERING MANHOURS

ENGINEERING MAN-HOURS ESTIMATE SUMMARY									
	PROJECT PRINCIPAL	PROJECT MANAGER	PROJECT ENGINEER	SENIOR ENGINEER	• _____ ENGINEER	ENGINEER TECHNICIAN	DRAFT	CLERICAL	TOTAL
1. Prelim. Design									
H&H Report (If Applicable)									
Alt. Loc. & Br. Type Study									
2. Final Design									
TS&L									
Foundation									
Final Plan									
Page Number in the proposal where all bridge sheets with titles are listed: _____									
3. PS&E									
4. Fatigue Analysis									
5. Seismic Analysis									
6. Load Capacity Analysis									
7. Other Special Item: _____									
TOTAL									
NOTE: Some classifications may not be applicable for each task.									

D-501 (5-12)



QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES (New Structures)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. BRIDGE GEOMETRY

Horizontal Geometry: _____

Width: Out-Out _____; Curb-Curb _____; Sidewalk _____

Skew: _____ (if < 70°, why?): _____

Minimum Clearance: Vertical → Provided = _____; Required = _____

Horizontal → Provided = _____; Required = _____

3. SUPERSTRUCTURE

Type: _____

Design Methodology: _____

Analysis Methodology (DM-4 Sec. 4): _____

Fatigue Design Roadway Class: _____

Fracture-Critical Elements: _____

No. of Spans: _____ Span Length(s): _____

No. of Beams: _____ (if > 4, why?): _____

Straight, Curved, or Kinked: _____

Beam Spacing: _____ Max. Overhang: _____

Web Size: _____ Slab Thickness: _____

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	_____	_____	_____	_____	_____	_____
Type:	_____	_____	_____	_____	_____	_____

D-501 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

DECK JOINTS	LOCATION	TYPE	MOVEMENT CLASSIFICATION
No. _____	_____	_____	_____
_____	_____	_____	_____
_____	_____	_____	_____

(if > 2, why?): _____

4. SUBSTRUCTURE

UNIT	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Type: _____	_____	_____	_____	_____	_____	_____
Support: _____	_____	_____	_____	_____	_____	_____
Geology: _____	_____	_____	_____	_____	_____	_____
# Borings: _____	_____	_____	_____	_____	_____	_____

(Note: Type = Structure Type; Support = Shallow or Deep Foundation; Geology = Type of Geological Formation)

5. MISCELLANEOUS

Are there any Superbeams? Yes ☐ No ☐

If yes, has the hauling permit been issued? Yes ☐ No ☐

Drainage: On Structure Number: _____ and Type _____

Off Structure Number: _____ and Type _____

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

Inspection Facilities: Yes ☐, and Type _____; or No ☐

Utilities: _____

Bridge-Mounted Sign Structures: Yes ☐ No ☐

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? _____

D-501 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

6. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Description:	_____	_____	_____	_____
Total Cost:	_____	_____	_____	_____
Calculation Page:	_____	_____	_____	_____

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: _____

7. ALTERNATES

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Superstructure:				
Type:	_____	_____	_____	_____
No. of Spans:	_____	_____	_____	_____
Beam Spacing:	_____	_____	_____	_____
No. of Beams:	_____	_____	_____	_____
Deck Quantity:	_____	_____	_____	_____
Deck Cost	_____	_____	_____	_____
Beam Quantity:	_____	_____	_____	_____
Beam Cost:	_____	_____	_____	_____

D-501 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Substructure:				
Abutment:				
Type:	_____	_____	_____	_____
Quantity:	_____	_____	_____	_____
Cost:	_____	_____	_____	_____
Pier:				
Type:	_____	_____	_____	_____
Quantity:	_____	_____	_____	_____
Cost:	_____	_____	_____	_____
Foundation:				
Type:	_____	_____	_____	_____
Quantity:	_____	_____	_____	_____
Cost:	_____	_____	_____	_____
TOTAL COST:	_____	_____	_____	_____

8. COMMENTS

D-502 (5-12)



QUALITY ASSURANCE FORM FOR T.S.&L. OF STEEL BRIDGES (Rehabilitation Structures)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

Structure Identification: _____

S-No.: _____

Bridge Type: _____ Year Built: _____

Design ADT: _____ Year: _____ ADTT Present: _____ Cumulative: _____

2. BRIDGE GEOMETRY

Proposed Width: Out-Out _____; Curb-Curb _____; Sidewalk _____

Skew: _____

Minimum Clearance: Vertical → Provided = _____; Required = _____

Horizontal → Provided = _____; Required = _____

3. SUPERSTRUCTURE

Structure to be Replaced: Deck ☐ Deck and Beams ☐ None ☐

Deck Condition Rating from NBIS report: _____ Year: _____

Proposed Deck Rehabilitation: _____

Calculation Page for Logic: _____

Design Methodology: _____

Analysis Methodology (DM-4 Sec. 4): _____

Remaining Fracture-Critical Elements: _____

Remaining Fatigue Life: _____ Calculation Page: _____

Retrofitted Distortion-Induced Fatigue Details: _____

D-502 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

No. of Spans: _____ Span Length(s): _____

No. of Beams: _____ (If > 4, Why?): _____

Straight, Curved, or Kinked: _____

Spacing: _____ Max. Overhang: _____

Web Size: _____ Slab Thickness: _____

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	_____	_____	_____	_____	_____	_____
Type:	_____	_____	_____	_____	_____	_____

DECK JOINTS	LOCATION	TYPE	MOVEMENT CLASSIFICATION
No. _____	_____	_____	_____
_____	_____	_____	_____
_____	_____	_____	_____

(if > 2, why?): _____

4. SUBSTRUCTURE

Parallel Eccentricity Limit _____; Calc. Page _____

Maximum Parallel Eccentricity _____; Calc. Page _____

Perpendicular Eccentricity Limit _____; Calc. Page _____

Maximum Perpendicular Eccentricity _____; Calc. Page _____

Sliding Resistance _____; Maximum Horiz. Load _____; Calc. Page _____

Bearing Resistance _____; Maximum Bearing Pressure _____; Calc. Page _____

Type of Scour Protection Proposed: _____

Is Substructure Adequately Reinforced? Yes ☐ No ☐

Type of Seismic Retrofit Proposed: _____

D-502 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

5. RATINGS

Ratings after Rehabilitation, including Future Wearing Surface:

LOAD	PHL-93 (Factor)	H (Tons)	HS (Tons)	ML-80 (Tons)	P-82 (Tons)
I.R.	_____	_____	_____	_____	N/A
O.R.	_____	_____	_____	_____	_____

6. MISCELLANEOUS

Are there any Superbeams? Yes ☐ No ☐

If yes, has the hauling permit been issued? Yes ☐ No ☐

Drainage: On Structure Number: _____ and Type _____

Off Structure Number: _____ and Type _____

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

Inspection Facilities: Yes ☐ No ☐

Utilities: _____

Bridge-Mounted Sign Structures: Yes ☐ No ☐

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? _____

D-502 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)**7. ALTERNATES SUMMARY**Is this a Streamlined submission? Yes ☐ No ☐If yes, how many Alternates are required 1 ☐ 2 ☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐ None ☐

If other, specify: _____

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4

Total Cost:	_____	_____	_____	_____

Calculation Page:	_____	_____	_____	_____

8. COMMENTS

D-503 (5-12)



QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES (New Structures)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. BRIDGE GEOMETRY

Horizontal Geometry: _____

Proposed Width: Out-Out _____; Curb-Curb _____; Sidewalk _____

Skew: _____ (if < 70°, why?): _____

Minimum Clearance: Vertical → Provided = _____; Required = _____

Horizontal → Provided = _____; Required = _____

3. SUPERSTRUCTURE

Type: _____

Design Methodology: _____

Analysis Methodology (DM-4 Sec. 4): _____

No. of Spans: _____ Span Length(s): _____

No. of Beams: _____ (if > 4, why?): _____

Beam Strength (f'_c): _____

Beam Spacing: _____ Max. Overhang: _____

Beam Size: _____ Slab Thickness: _____

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	_____	_____	_____	_____	_____	_____
Type:	_____	_____	_____	_____	_____	_____

D-503 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

DECK JOINTS	LOCATION	TYPE	MOVEMENT CLASSIFICATION
No. _____	_____	_____	_____
_____	_____	_____	_____
_____	_____	_____	_____

(if > 2, why?): _____

4. SUBSTRUCTURE

UNIT	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Type: _____	_____	_____	_____	_____	_____	_____
Support: _____	_____	_____	_____	_____	_____	_____
Geology: _____	_____	_____	_____	_____	_____	_____
# Borings: _____	_____	_____	_____	_____	_____	_____

(Note: Type = Structure Type; Support = Shallow or Deep Foundation; Geology = Type of Geological formation)

5. MISCELLANEOUS

Are there any Superbeams? Yes ☐ No ☐

If yes, has the hauling permit been issued? Yes ☐ No ☐

Drainage: On Structure Number: _____ and Type _____

Off Structure Number: _____ and Type _____

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

Inspection Facilities: Yes ☐ , and Type _____; or No ☐

Utilities: _____

D-503 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Bridge-Mounted Sign Structures: Yes ☐ No ☐

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? _____

6. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Description:	_____	_____	_____	_____
Total Cost:	_____	_____	_____	_____
Calculation Page:	_____	_____	_____	_____

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: _____

7. ALTERNATES

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Superstructure:				
Type:	_____	_____	_____	_____
No. of Spans:	_____	_____	_____	_____
Beam Spacing:	_____	_____	_____	_____
No. of Beams:	_____	_____	_____	_____
Deck Quantity:	_____	_____	_____	_____
Deck Cost	_____	_____	_____	_____
Beam Quantity:	_____	_____	_____	_____
Beam Cost:	_____	_____	_____	_____

D-503 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Substructure:				
Abutment:				
Type:	_____	_____	_____	_____
Quantity:	_____	_____	_____	_____
Cost:	_____	_____	_____	_____
Pier:				
Type:	_____	_____	_____	_____
Quantity:	_____	_____	_____	_____
Cost:	_____	_____	_____	_____
Foundation:				
Type:	_____	_____	_____	_____
Quantity:	_____	_____	_____	_____
Cost:	_____	_____	_____	_____
TOTAL COST:	_____	_____	_____	_____

8. COMMENTS

D-504 (5-12)



QUALITY ASSURANCE FORM FOR T.S.&L. OF PRESTRESSED CONCRETE BRIDGES (Rehabilitation Structures)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____

Bridge Type: _____ Year Built: _____

Design ADT: _____ Year: _____ ADTT Present: _____ Cumulative: _____

2. BRIDGE GEOMETRY

Proposed Width: Out-Out _____; Curb-Curb _____; Sidewalk _____

Skew: _____

Minimum Clearance: Vertical → Provided = _____; Required = _____

Horizontal → Provided = _____; Required = _____

3. SUPERSTRUCTURE

Structure to be Replaced: Deck ☐ Deck and Beams ☐ None ☐

Deck Condition Rating from NBIS report: _____ Year: _____

Proposed Deck Rehabilitation: _____

Calculation Page for Logic: _____

Design Methodology: _____

Analysis Methodology (DM-4 Sec. 4): _____

No. of Spans: _____ Span Length(s): _____

No. of Beams: _____ (if > 4, why?): _____

Beam Spacing: _____ Max. Overhang: _____

Beam Size: _____ Slab Thickness: _____

D-504 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

BEARINGS	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3	PIER 4
Function:	_____	_____	_____	_____	_____	_____
Type:	_____	_____	_____	_____	_____	_____
DECK JOINTS	LOCATION	TYPE		MOVEMENT CLASSIFICATION		
No. _____	_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____	_____

(if > 2, why?): _____

4. SUBSTRUCTURE

Parallel Eccentricity Limit _____; Calc. Page _____

Maximum Parallel Eccentricity _____; Calc. Page _____

Perpendicular Eccentricity Limit _____; Calc. Page _____

Maximum Perpendicular Eccentricity _____; Calc. Page _____

Sliding Resistance _____; Maximum Horiz. Load _____; Calc. Page _____

Bearing Resistance _____; Maximum Bearing Pressure _____; Calc. Page _____

Type of Scour Protection Proposed: _____

Is Substructure Adequately reinforced? Yes ☐ No ☐

Type of Seismic Retrofit Proposed: _____

5. RATINGS

Ratings after Rehabilitation, including Future Wearing Surface:

LOAD	PHL-93 (Factor)	H (Tons)	HS (Tons)	ML-80 (Tons)	P-82 (Tons)
I.R.	_____	_____	_____	_____	N/A
O.R.	_____	_____	_____	_____	_____

D-504 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

6. MISCELLANEOUS

Are there any Superbeams? Yes ☐ No ☐

If yes, has the hauling permit been issued? Yes ☐ No ☐

Drainage: On Structure Number: _____ and Type _____

Off Structure Number: _____ and Type _____

Protective Fence: Yes ☐ No ☐

Navigation Lights: Yes ☐ No ☐

Inspection Facilities: Yes ☐, and Type _____; No ☐

Utilities: _____

Bridge-Mounted Sign Structures: Yes ☐ No ☐

Bridge-Mounted Lighting: Yes ☐ No ☐

Was High Mast or Pier-Mounted Considered? Yes ☐ No ☐ If no, why? _____

7. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: _____

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Total Cost:	_____	_____	_____	_____
Calculation Page:	_____	_____	_____	_____

D-504 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

8. COMMENTS

D-505 (5-12)



QUALITY ASSURANCE FORM FOR FOUNDATIONS

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Skew: _____ Grade: _____

Bridge Type: _____

No. of Substructure Units: _____

Note: **Bold-faced** parameters are those requiring approval of Central Office BDTD.

2. FOUNDATION TYPE

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
Spread Footing on Rock: (Complete Sections 3 & 4)	_____	_____	_____	_____	_____
Spread Footing on Soil: (Complete Sections 3 & 5)	_____	_____	_____	_____	_____
Point Bearing Pile Size: (Complete Sections 3 & 6)	_____	_____	_____	_____	_____
End Bearing Pile Size: (Complete Sections 3 & 6)	_____	_____	_____	_____	_____
Friction Pile Size: (Complete Sections 3 & 6)	_____	_____	_____	_____	_____
Caisson Size: (Complete Sections 3 & 6)	_____	_____	_____	_____	_____
Pedestal Size: (Complete Section 3)	_____	_____	_____	_____	_____
Depth of Footing Embedment**:	____/____	____/____	____/____	____/____	____/____

Comments (Special Treatments, etc.) _____

See page 6 of 7 for notes

D-505 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

3. BEARING STRATUM AND FOUNDATION PARAMETER INFORMATION

Any solution cavity, mining, pH, resistivity, sulfate, dissolved chloride, fill, or bacteria problems?

Yes ☐ No ☐

If yes, identify problem, location, and depth _____

	ABUT. 1	ABUT. 2	PIER1	PIER2	PIER 3*
Bearing Stratum:	_____	_____	_____	_____	_____
(1.5 x footing width below bottom of spread footings on soils) (15-pile dia. above tip elevation for end bearing & friction piles)					
RQD:	_____	_____	_____	_____	_____
% Gross Recovery:	_____	_____	_____	_____	_____
Average N Value (blows/ft):	_____	_____	_____	_____	_____
Unconfined Comp. Test Result (Rock or Soil) (tsf):	_____	_____	_____	_____	_____
Non-Saturated Soil Density (tcf):	_____	_____	_____	_____	_____
Saturated Soil Density (tcf):	_____	_____	_____	_____	_____
Soil Slope Inclination (if on slope) i (deg.):	_____	_____	_____	_____	_____
Depth from grade to groundwater, $D_f + Z_w$ (ft):	_____	_____	_____	_____	_____
Undrained Shear Strength, S_u (tsf):	_____	_____	_____	_____	_____
Foundations & Loadings					
V^{***} (tons):	_____	_____	_____	_____	_____
H^{***} (tons):	_____	_____	_____	_____	_____
Bottom of Footing Elevation (ft):	_____	_____	_____	_____	_____
D_f (ft.):	_____	_____	_____	_____	_____
B (ft):	_____	_____	_____	_____	_____
L (ft):	_____	_____	_____	_____	_____

See page 6 of 7 for notes

D-505 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

4. SPREAD FOOTINGS - ON ROCK BEARING STRATUM

				ABUT. 1	ABUT. 2	PIER1	PIER2	PIER 3*
Coefficient of Friction between rock & footing:				_____	_____	_____	_____	_____
Show the appropriate information for A or B	(A) Semi-Empirical Method	Use a or b	(a) RMR:	_____	_____	_____	_____	_____
			(b) NGI:	_____	_____	_____	_____	_____
			C _o (tsf):	_____	_____	_____	_____	_____
			N _{ms} :	_____	_____	_____	_____	_____
	(B) Analytic Method		c (tsf):	_____	_____	_____	_____	_____
			ϕ_{fm} :	_____	_____	_____	_____	_____
			Horiz. Joint Spacing, S (ft):		_____	_____	_____	_____
			Closely Spaced Joints (D10.6.3.2.3aP)	Open Joints	_____	_____	_____	_____
		K _e :		_____	_____	_____	_____	
		Closed Joints		N _c :	_____	_____	_____	_____
			N _q :	_____	_____	_____	_____	
			N _y :	_____	_____	_____	_____	
		Y (tcf):	_____	_____	_____	_____	_____	
		Widely Spaced Joints (D10.6.3.2.3bP)	J:	_____	_____	_____	_____	_____
	N _{cr} :		_____	_____	_____	_____	_____	
	Vert. Joint Spacing, H _p (ft.):		_____	_____	_____	_____	_____	
K _e :	_____		_____	_____	_____	_____		
q_{ult} -STRN (tsf):				_____	_____	_____	_____	_____
Sliding Resistance Factor, ϕ_r:				_____	_____	_____	_____	_____
Bearing Resistance Factor, ϕ_b:				_____	_____	_____	_____	_____
q_o – SERV at anticipated settlement (tsf):				_____	_____	_____	_____	_____
Estimated Total Settlement (in.):				_____	_____	_____	_____	_____
Tolerable Settlement (in.):				_____	_____	_____	_____	_____

For rocks defined as very poor quality in the Semi-Empirical Method, complete Section 5 with an equivalent soil mass
 See page 6 of 7 for notes

D-505 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

5. SPREAD FOOTINGS - ON SOIL BEARING STRATUM

			ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
Coefficient of Friction between soil & footing:			_____	_____	_____	_____	_____
Show the appropriate information for A, B or C	(A) Theoretical Estimation Method:	c (tsf):	_____	_____	_____	_____	_____
	(If on slope, substitute N_{cq} & N_{yq} for N_c and N_y)	ϕ_{fm}:	_____	_____	_____	_____	_____
		N_y:	_____	_____	_____	_____	_____
		N_c:	_____	_____	_____	_____	_____
		N_q:	_____	_____	_____	_____	_____
	(B) SPT Method:	$\bar{N}_{160}$:	_____	_____	_____	_____	_____
	(C) CPT Method:	q_c:	_____	_____	_____	_____	_____
q_{ult} -STRN (tsf):			_____	_____	_____	_____	_____
Sliding Resistance Factor, ϕ_r:			_____	_____	_____	_____	_____
Bearing Resistance Factor, ϕ_b:			_____	_____	_____	_____	_____
q_o – SERV at anticipated settlement (tsf):			_____	_____	_____	_____	_____
Estimated Total Settlement (in.):			_____	_____	_____	_____	_____
Estimated Settlement After Beam Erection (in.):			_____	_____	_____	_____	_____
Tolerable Net Settlement (in.):			_____	_____	_____	_____	_____

See page 6 of 7 for notes

D-505 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

6. PILE, DRILLED SHAFT, AND MICROPILE FOUNDATIONS

				ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
Static Analysis Method:				_____	_____	_____	_____	_____
Pile Size (e.g., HP 12 x 74) or Pile/Shaft Diameter (ft.):				_____	_____	_____	_____	_____
Micropile drill hole diameter through bonded length (ft.):				_____	_____	_____	_____	_____
Pile/Shaft Length (ft.):				_____	_____	_____	_____	_____
Rock Socket Depth/Micropile bonded length (ft.):				_____	_____	_____	_____	_____
Show the appropriate information for A, B or C (friction and end-bearing piles and drilled shafts)	(A) Semi-Empirical Method:	Show the appropriate information for a, b, c or d	(a) Nordlund					
			K_δ :	_____	_____	_____	_____	_____
			P_d (tsf):	_____	_____	_____	_____	_____
			C_F :	_____	_____	_____	_____	_____
			δ (tsf):	_____	_____	_____	_____	_____
		(b)	α :	_____	_____	_____	_____	_____
		(c)	β :	_____	_____	_____	_____	_____
		σ'_v (tsf):	_____	_____	_____	_____	_____	
		(d)	λ :	_____	_____	_____	_____	_____
		σ'_v (tsf):	_____	_____	_____	_____	_____	
		(B) SPT Method:	N_{160} :	_____	_____	_____	_____	_____
		$\bar{N}_{160}$:	_____	_____	_____	_____	_____	
	D_b (ft.):	_____	_____	_____	_____	_____		
	D (ft.):	_____	_____	_____	_____	_____		
	σ'_v (tsf):	_____	_____	_____	_____	_____		
	(C) CPT Method: (Requires approval of Chief Bridge Engineer)	q_{c1} (tsf):	_____	_____	_____	_____	_____	
	q_{c2} (tsf):	_____	_____	_____	_____	_____		
	Q_s (tsf):	_____	_____	_____	_____	_____		
Micropile grout-to-ground bond strength, α_b (ksi):				_____	_____	_____	_____	_____

See page 6 of 7 for notes

D-505 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

6. PILE, DRILLED SHAFT, AND MICROPILE FOUNDATIONS (continued)

	ABUT. 1	ABUT. 2	PIER 1	PIER 2	PIER 3*
Shaft Resistance, Q_s (tsf):	_____	_____	_____	_____	_____
Tip Resistance (tsf):	_____	_____	_____	_____	_____
Total Resistance (tsf):	_____	_____	_____	_____	_____
Side Resistance (tsf):	_____	_____	_____	_____	_____
ϕ ****:	_____	_____	_____	_____	_____
Group Resistance (tsf):	_____	_____	_____	_____	_____
Is pile buckling a consideration?:	_____	_____	_____	_____	_____
Pile Spacing (ft.):	_____	_____	_____	_____	_____
Maximum Factored Vertical Load per Pile****(kips):	_____	_____	_____	_____	_____
Unfactored (Service I) Vertical Load per Pile (kips):	_____	_____	_____	_____	_____
Maximum Factored Lateral Load per Pile**** (kips):	_____	_____	_____	_____	_____
Unfactored (Service I) Lateral Load per Pile (kips):	_____	_____	_____	_____	_____
Estimated Total Settlement (in.):	_____	_____	_____	_____	_____
Estimated Lateral Settlement (in.):	_____	_____	_____	_____	_____
Estimated Settlement Before Beam Erection (in.):	_____	_____	_____	_____	_____
Tolerable Settlement (in.):	_____	_____	_____	_____	_____

* Use additional sheets if more than three piers.

** From the existing groundline to the bottom of the footing/From the finished groundline to the bottom of the footing.

*** H and V values are those from the load combination which controlled the strength design for bearing pressure.

**** Maximum factored pile loads are those from the load combination which controlled the strength design for bearing pressure.

***** ϕ may be increased to a value between that shown in Table D10.5.5.2.3-1 for the method used to calculate total resistance and the value shown for "load tests" based on the data in DM-4, Appendix F, if it applies to the site conditions under consideration.

7. SCOUR INFORMATION

Stream X-section at Bridge shown on Calculation Page: _____

Value for projection into channel: A _____

Average Approach Velocity: Q_{100} = _____ ft./sec.: Q_{500} = _____ ft./sec.

Scour Computations Based on: FHWA Tech. Advisory ☐ HEC-18 ☐

D-505 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Computed Scour Depths:

ITEMS		SCOUR DEPTH IN INCHES									
		ABUT. 1		ABUT. 2		PIER 1		PIER 2		PIER 3*	
		Q ₁₀₀	Q ₅₀₀	Q ₁₀₀	Q ₅₀₀	Q ₁₀₀	Q ₅₀₀	Q ₁₀₀	Q ₅₀₀	Q ₁₀₀	Q ₅₀₀
AGGRADATION OR DEGRADATION SCOUR											
CONTRACTION SCOUR											
LOCAL SCOUR											
TOTAL SCOUR											
PROPOSED FOOTING ELEVATION	TOP										
	BOTTOM										
ADJACENT STREAM BED ELEVATION											
PROPOSED PILE OR SHAFT TIP ELEVATION											

Any known scour problems at or near the location of the proposed substructure unit(s)?

Yes ☐ No ☐

If yes, give pertinent information, such as scour depths: _____

Scour protection reduction utilized? Yes ☐ No ☐

*Use additional sheets if more than three piers.

D-506 (5-12)



QUALITY ASSURANCE FORM FOR GLULAM TIMBER BRIDGES

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. BRIDGE GEOMETRY

Horizontal Geometry: _____

Width: Out-Out _____; Curb-Curb _____; Sidewalk _____

Span Length: _____

Skew: _____ (if < 70°, why?): _____

Minimum Clearance: Vertical → Provided = _____; Required = _____

Horizontal → Provided = _____; Required = _____

3. SUPERSTRUCTURE

Type: Glulam Longitudinal Panel ☐ Glulam Deck/Steel Beams ☐ Glulam Deck/Glulam Beams ☐

Design Live Load: PHL-93 ☐ H _____ ☐ HS _____ ☐ ML-80 ☐ P-82 ☐

Longitudinal Panel Size: _____

Max. Flexural Stress in Longitudinal Panel: _____ ksi,

Flexural Resistance: _____ ksi (wet usage); Calc. Page: _____

Species of Hardwood: _____; Grade: _____

Deck Panel Interconnecting Device: Dowel ☐ Other ☐, specify _____

Beam Size: _____ Beam Spacing: _____

Max. Flexural Stress in Beam: _____ ksi,

Flexural Resistance: _____ ksi (wet usage); Calc. Page: _____

Species of Hardwood: _____; Grade: _____

L.L. Deflection: Allowed = $L/425$ = _____; Designed: _____

Calc. Page: _____

D-506 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

4. SUBSTRUCTURE

Abut. Type: _____; Bot. of Ftg. Elev.: _____; Scour Depth Elev.: _____

Pile Type (Material) _____; Est. Pile Tip Elev. _____;

Bot. of Lagging Elev. _____; *Average N _____

Pier Type: _____; Bot. of Ftg. Elev.: _____; Scour Depth Elev.: _____

Pile Type (Material) _____; Est. Pile Tip Elev. _____;

*Average N _____

Anticipated Debris Problem: Yes ☐ No ☐

*If timber piles are proposed, average N value is for the strata above the bearing stratum.

5. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: _____

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Total Cost:	_____	_____	_____	_____
Calculation Page:	_____	_____	_____	_____

6. COMMENTS

D-507 (5-12)



QUALITY ASSURANCE FORM FOR STRESSED TIMBER BRIDGES

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. BRIDGE GEOMETRY

Horizontal Geometry: _____

Width: Out-Out _____; Curb-Curb _____; Sidewalk _____

Span Length: _____

Skew: _____ (if < 70°, why?): _____

Minimum Clearance: Vertical → Provided = _____; Required = _____

Horizontal → Provided = _____; Required = _____

3. SUPERSTRUCTURE

Design Live Load: PHL-93 ☐ H _____ ☐ HS _____ ☐ ML-80 ☐ P-82 ☐

Species of Hardwood: _____; Grade: _____

Composite Stressed Timber Design: Yes ☐ No ☐

If yes, spacing of galvanized steel sandwich plates _____ in. center-to-center

Max. Flexural Stress in Deck: _____ ksi

Flexural Resistance: _____ ksi (wet usage); Calc. Page: _____

L.L. Deflection: Allowed = $L/425$ = _____; Designed: _____

Calc. Page: _____

Does butt joint pattern meet criteria: Yes ☐ No ☐

Size of channel anchorage: _____

Ultimate tensile strength of prestressing rods: _____ ksi

Is triple corrosion protection specified: Yes ☐ No ☐

Minimum prestress between laminations: _____ ksi

Interval of retensioning: 2nd ☐ 3rd ☐ 4th ☐ days

D-507 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

4. SUBSTRUCTURE

Abut. Type: _____; Bot. of Ftg. Elev.: _____; Scour Depth Elev.: _____

Pile Type (Material) _____; Est. Pile Tip Elev. _____;

Bot. of Lagging Elev. _____; *Average N _____

Pier Type: _____; Bot. of Ftg. Elev.: _____; Scour Depth Elev.: _____

Pile Type (Material) _____; Est. Pile Tip Elev. _____;

*Average N _____

Anticipated Debris Problem: Yes ☐ No ☐

*If timber piles are proposed, average N value is for the strata above the bearing stratum.

5. ALTERNATES SUMMARY

Is this a Streamlined submission? Yes ☐ No ☐

If yes, how many Alternates are required? 1 ☐ 2 ☐

Permitted Alternates by Contractors:

Prestressed Concrete ☐ Steel ☐ Other ☐

If other, specify: _____

ITEM DESCRIPTION	ALTERNATE #1	ALTERNATE #2	ALTERNATE #3	ALTERNATE #4
Total Cost:	_____	_____	_____	_____
Calculation Page:	_____	_____	_____	_____

6. COMMENTS

D-508 (5-12)



QUALITY ASSURANCE FORM FOR COMPOSITE STEEL-GIRDER SUPERSTRUCTURE DESIGN

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. GENERAL INFORMATION

No. of Spans: _____; Radius of Curvature: _____; Max. Skew: _____

Span Length(s) and Arrangement: _____

No. of Girders: _____; Girder Spacing: _____; Web Depth: _____

Any deviations from T.S.&L. and Foundation approvals? Yes ☐ No ☐

If yes, indicate reasons: _____

List restrictions on Alternate Design, if any: _____

Design Method: _____; Analysis Method: _____

3. SUPERSTRUCTURE

Section Properties used for Calculation of Moments, Shears, and Deflections:

Max. Positive Moment $I =$ _____; Calc. Page _____

Max. Negative Moment $I =$ _____; Calc. Page _____

Section Properties used for Calculation of Girder Stresses:

Max. Positive Moment $I =$ _____; Calc. Page _____

Max. Negative Moment $I =$ _____; Calc. Page _____

Greatest Girder Moment and which Live Load Governed: Interior ☐ Exterior ☐

Max. Positive Moment $M =$ _____; Live Load _____; Calc. Page _____

Max. Negative Moment $M =$ _____; Live Load _____; Calc. Page _____

D-508 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Greatest locked-in girder stresses (compression and tension) due to deck placement sequence:

Magnitude _____; Calc. Page _____

Location _____; Calc. Page _____

Resistance _____; Calc. Page _____

Prior to deck hardening:

Max. Comp. non-composite flange stress _____; Calc. Page _____

Location _____; Calc. Page _____

Resistance _____; Calc. Page _____

Max. Comp. non-composite web buckling stress _____; Calc. Page _____

Location _____; Calc. Page _____

Resistance _____; Calc. Page _____

Vertical placement of temporary overhang support bracket
 (in terms of the depth of the girder web) _____; Calc. Page _____

Max. transverse stiffener spacing _____; Calc. Page _____

Max. service live-load + impact deflection as a ratio of span/deflection:

Deflection _____; Calc. Page _____

Location _____; Calc. Page _____

Load path chosen to carry transverse wind load to the bearings:

_____; Calc. Page _____

Category of the least fatigue resistant detail used _____; Calc. Page _____

Stress range at the most critical of these details _____; Calc. Page _____

Location of most critical detail _____; Calc. Page _____

Ultimate fatigue resistance _____; Calc. Page _____

Diaphragm design method _____; Calc. Page _____

D-508 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

Bearing Types:

Expansion _____ ; Calc. Page _____

Fix _____ ; Calc. Page _____

Expansion Dam Type _____ ; Calc. Page _____

Expansion Movement _____ ; Calc. Page _____

D-509 (5-12)



QUALITY ASSURANCE FORM FOR PRESTRESSED CONCRETE BRIDGE DESIGN

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. SUPERSTRUCTURE

No. of spans: _____; Span length(s): _____; Max. Skew: _____

Type and Size of Beams: _____

Any deviations from T.S.&L. and Foundation approvals? Yes ☐ No ☐

If yes, indicate reasons _____

List restrictions on Alternate Design, if any: _____

If multi-span, is it a jointless design? Yes ☐ No ☐ N/A ☐

If no, indicate reasons: _____

LL Distribution: PennDOT Approximate ☐ Finite Element ☐ Other ☐

Strands: Size _____

270 ksi Low Lax _____; Stress Rel _____; Straight _____; Draped _____

Debonded in lieu of draping _____; % Debonded _____

Max. Unfact. Pos. Moment _____ kip-ft. (Calc. Page _____)

Location _____

Max. Unfact. Neg. Moment (Slab @ Cont.) _____ kip-ft. (Calc. Page _____)

Location _____

Final Tensile Stresses in Precomp. Tensile Zone:

PHL-93 (actual) = _____ ksi (Calc. Page _____)

Resistance = _____ ksi (Calc. Page _____)

D-509 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Maximum Prestressing Force _____ kips; Eccentricity _____ in.

For Critical Section*: Maximum Factored Flexural Resistance (M_r) _____ kip-ft.;

Maximum Factored Moment (M_u) _____ kip-ft.

*Based on controlling vehicle, either PHL-93 or P-92.

Tension Controlled: _____; Compression Controlled: _____

Does Web Thickness meet the I-Beam minimum of 8 in.? Yes ☐ No ☐
 {except 18/30, 18/33 and 18/36 beams, which is 6 in.}

Does Top Flange Thickness meet AASHTO Type V/VI Beam (minimum of 5 in)? Yes ☐ No ☐

Does Top Flange Thickness meet PA Bulb-Tee Beam (minimum of 4 1/2 in)? Yes ☐ No ☐

Transverse Tendon Layout as per BD-651M and BC-775M? Yes ☐ No ☐ N/A ☐

Deck Placement Sequence shown for Cont. Spans show on sheet: _____ (including diaphragm area)

If Draped I-Beam Design:

Is extra Shear Steel furnished at drape point? Yes ☐ No ☐

Is Epoxy-Coated Reinforcement provided:

For 9 ft. length at ends of all beams adjacent to joints? Yes ☐ No ☐

Recessed Strand Detail Shown in Plans on Sheet _____

Are Strands Debonded in Bottom Row? Yes ☐ No ☐

For continuous designs: Are shear and moment envelopes or tables shown? Yes ☐ No ☐

Is positive moment reinforcement provided/required? Yes ☐ No ☐

D-509 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

3. SUBSTRUCTURE

Bearing Types:

Expansion: _____ (Calc. Page _____)

Fix: _____ (Calc. Page _____)

Expansion Dam Type: _____

Design Load for Shear Blocks: _____ (Calc. Page _____)

Was Live Load Considered in Design of Backwall as per DM-4? Yes ☐ No ☐ N/A ☐Are Beam Seats at both substructure units sloped the same for adjacent Box Beams? Yes ☐ No ☐ N/A ☐

(Use camber values for longitudinal slopes)

4. COMMENTS

D-510 (5-12)



QUALITY ASSURANCE FORM FOR ELASTOMERIC BEARING DESIGN

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

Calculation Page: _____

2. **DUROMETER** 50 _____; 60 _____; Other: _____

3. **MINIMUM SHAPE FACTOR** _____

4. COMPRESSIVE STRESS

(a) Calculated Maximum Compressive Stress = _____ ksi

(b) Compressive Stress Resistance = _____ ksi

5. **MAXIMUM CALCULATED COMPRESSIVE DEFLECTION** _____ in.

6. ROTATION

(a) Construction Tolerance About Longitudinal Axis: _____ radians (for spans ≥ 100 ft.)

(b) Total Computed Maximum Rotation About Longitudinal Axis: _____ radians

(c) Construction Tolerance About Transverse Axis: _____ radians

(d) Total Computed Maximum Rotation About Transverse Axis: _____ radians

7. BEARING PAD THICKNESS

Plain Bearing Pad:

(a) Maximum bearing pad thickness used on this bridge: _____ in.

(b) Minimum bearing pad thickness used on this bridge: _____ in.

Laminated Bearing Pad:

(a) Maximum inner layer thickness: _____ in.

(b) Minimum inner layer thickness: _____ in.

(c) Total Maximum Height: _____ in.

8. SHEAR

Calculated Maximum shear deflection = _____ in.

D-510 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)**9. STABILITY**

(a) For Plain Rectangular Pad

$$\frac{\text{Smallest Dimension}}{5} = \text{_____ in.} \geq \text{Maximum Height of Bearing}$$

(b) For Reinforced Rectangular Bearing Pad

$$\frac{\text{Smallest Dimension}}{3} = \text{_____ in.} \geq \text{Maximum Height of Bearing}$$

(c) For Plain Circular Pad

$$\frac{\text{Diameter}}{6} = \text{_____ in.} \geq \text{Maximum Height of Bearing}$$

(d) For Reinforced Circular Bearing Pad

$$\frac{\text{Diameter}}{4} = \text{_____ in.} \geq \text{Maximum Height of Bearing}$$

10. BEARING SEAT SLOPESlope due to net camber and roadway grade in the direction parallel to the longitudinal axis of the beam
= _____ %

(If exceeds 1%, accommodate the remaining slope in the design of the bearing pad or by modifying the bottom of beam area.)

Slope of the bottom of the beam in the direction parallel to the transverse axis of the bearing pad
= _____ %

(For box beams, provide temporary lateral support to the beam during construction until the end diaphragms are cast and the shear blocks or dowel bars are installed if slope exceeds 5% and pads thicker than 3.5 in. are used.)

D-511 (5-12)



QUALITY ASSURANCE FORM FOR HIGH LOAD MULTI-ROTATIONAL BEARING DESIGN

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. BEARING TYPES

Pot _____; Spherical _____; Disc _____; Other/Specify _____

If other, specify the specification used: _____

3. REACTIONS USED FOR FIXED BEARINGS

Location: _____; Calc. Page _____

Controlling Limit State _____

Maximum Reaction:

Non-Seismic: Vertical: _____ kips; Horizontal: _____ kips

Seismic: Vertical: _____ kips; Horizontal: _____ kips

Maximum Total Horizontal Reaction at One (1) Substructure Unit: _____ kips

4. REACTIONS USED FOR EXPANSION BEARINGS

Location: _____; Calc. Page _____

Maximum Reaction: Vertical: _____ kips; Horizontal: _____ kips

Minimum Reaction: Vertical: _____ kips; Horizontal: _____ kips

Minimum Number of Guided Bearings: _____

5. MAXIMUM MOVEMENT

Thermal Expansion = _____ in.; Thermal Contraction = _____ in.

Camber Changes = _____ in.; Creep and Shrinkage = _____ in.

Construction Tolerance = _____ in.

TOTAL Maximum Movement = _____ in.

D-511 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

6. POT BEARINGS

	FIXED	GUIDED/ EXPANSION
Max. Bearing Pressure on Pot Wall*	_____	_____
Max. Principal Stress in Pot Wall due to hoop, bending, shear, and compression stresses*	_____	_____
Stress Resistance in Pot Wall*	_____	_____
Stress Resistance in Guide Key*	_____	_____
Design Rotation	_____	_____
Depth of Pot Cavity	_____	_____
Max. Thickness of Pot Beneath Elastomer	_____	_____
Min. Thickness of Pot Beneath Elastomer	_____	_____
Thickness of Elastomeric Disc	_____	_____
Min. Thickness of Elastomeric Disc	_____	_____
Clear Between Top of Pot and Sliding Surface	_____	_____
Min. Clear Between Top of Pot and Sliding Surface	_____	_____

7. CONFIGURATION

Maximum size in in. and	_____	_____
Number of Sealing rings*	_____	_____
Minimum size in in. and	_____	_____
Number of Sealing rings*	_____	_____

*N/A if BD-613M is used.

8. SPHERICAL BEARINGS

To be developed at a later date.

Calc. Page _____

9. DISC BEARINGS

To be developed at a later date.

Calc. Page _____

D-512 (5-12)



QUALITY ASSURANCE FORM FOR PIPES AND CULVERTS

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. CULVERT TYPE AND GEOMETRY

CULVERT TYPE:	CONCRETE PIPE	REINFORCED CONC. BOX	PLATE PIPE	PLATE PIPE ARCH	METAL BOX
Fill Height:	_____	_____	_____	_____	_____
Skew:	_____	_____	_____	_____	_____
Length:	_____	_____	_____	_____	_____
Size:	_____	_____	_____	_____	_____

Is fish passage provided? Yes ☐ No ☐ N/A ☐

3. GENERAL

Backfill Unit Density: _____

Railroad Live Load: Yes ☐ No ☐

pH: Foundation Material (Rock/Soil) _____; Water _____

Method of Corrosion Protection: _____

Anticipated removal of unsuitable material: Yes ☐ No ☐

Anticipated Settlement: _____; Calculation Page _____

Location	Inlet	Outlet
Type of End Wall Provided:	_____	_____
Type of Scour Protection Provided:	_____	_____

D-512 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)**4. DESIGN REQUIREMENTS**

Design Life: _____ ; Calculation Page _____

Concrete Pipes:Design Method: ☐ BD-636M; ☐ Other, specify _____Installation Method: ☐ RC-30M; ☐ Other, specify _____

Method of Abrasion Protection: _____

Concrete Box Culverts:

Precast _____ ; Cast-in-Place (CIP) _____

(If CIP, why?) _____

Computer program used: _____

Method of Abrasion Protection: _____

Metal Culverts:

Design Spec.: _____

How is Design Life accounted for: Additional Metal ☐ Concrete Paving ☐Relieving Slab Provided: Yes ☐ No ☐Tension Strut Required: Yes ☐ No ☐**5. ALTERNATES**

Permitted Alternates by Contractor: _____

6. COMMENTS

D-513 (5-12)



QUALITY ASSURANCE FORM FOR BRADD FINAL PLANS

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. GENERAL

(a) BRADD Version No.: _____

(b) Which Designer Notes from Sheet Zero are not applicable? _____

Give reasons: _____

(c) Do details in contract drawings agree with current supplemental drawings? Yes ☐ No ☐

If no, justify: _____

(d) Indicate controlling rating Moment (M) or Shear (S), with FWS.

LOADING

HL-93

PHL-93

Rating:
(Factor) IR ___ M ___ S ___

IR ___ M ___ S ___

OR ___ M ___ S ___

OR ___ M ___ S ___

LOADING

HS

ML-80

P-82

Rating:
(Tons) IR ___ M ___ S ___

IR ___ M ___ S ___

OR ___ M ___ S ___

OR ___ M ___ S ___

OR ___ M ___ S ___

3. SUPERSTRUCTURE

(a) Beam Data: Beam Type: _____ ; No. of Beams: _____

Beam Spacing: _____ ; Max. Overhang: _____

(b) Bearing Data: Fixed Expansion

Type: _____

Size: (W x L x H): _____

D-513 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)**4. SUBSTRUCTURE**(a) Type of Abutment: Stub ☐ High ☐ Wall ☐

(b) Abutment Stem Steel/Linear ft. _____

(c) Type of Pier(s): _____

5. OTHER

List all Non BRADD Details and Sheet Nos.

Detail	Sheet No.		Detail	Sheet No.
_____	_____		_____	_____
_____	_____		_____	_____
_____	_____		_____	_____

NOTE: Other pertinent Q/A form will apply and must be completed.

D-514 (5-12)



QUALITY ASSURANCE FORM FOR SUBSTRUCTURES

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Skew: _____ Span(s): _____

Superstructure Type: _____

2. SUBSTRUCTURE TYPE

	ABUT. 1	PIER 1	PIER 2	PIER 3*	ABUT. 2
Unit Type:	_____	_____	_____	_____	_____
Design Height:	_____	_____	_____	_____	_____
Footing Width:	_____	_____	_____	_____	_____
Footing Length:	N/A	_____	_____	_____	N/A
Footing Thickness:	_____	_____	_____	_____	_____
Parallel Eccentricity Limit:	_____	_____	_____	_____	_____
Maximum Parallel Eccentricity:	_____	_____	_____	_____	_____
Perpendicular Eccentricity Limit:	_____	_____	_____	_____	_____
Maximum Perpendicular Eccentricity:	_____	_____	_____	_____	_____
Maximum Horizontal Load:	_____	_____	_____	_____	_____
Sliding Resistance:	_____	_____	_____	_____	_____
Beam Seat Length:	_____	_____	_____	_____	_____
Required:	_____	_____	_____	_____	_____
Provided:	_____	_____	_____	_____	_____
Superstructure Bearings:	_____	_____	_____	_____	_____
Fixity (E/F):	_____	____/____	____/____	____/____	_____
Type:	_____	____/____	____/____	____/____	_____

Comments: _____

D-514 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

3. FOUNDATION DATA

Date of Foundation Approval: _____

	ABUT. 1	PIER 1	PIER 2	PIER 3*	ABUT. 2
Foundation Type:	_____	_____	_____	_____	_____
Foundation (Load/Pressure)** Resistance:	_____	_____	_____	_____	_____
Maximum (Load/Pressure)** Design:	_____	_____	_____	_____	_____
Minimum (Load/Pressure)** Design:	_____	_____	_____	_____	_____
Comments:	_____				

*Use additional sheets if more than three piers.

**For foundations supported on piles, provide a load value;
 For foundations supported on spread footings, provide a pressure.

4. COMPUTER DESIGN PROGRAM

	ABUTMENT DESIGN	PIER DESIGN	SEISMIC ANALYSIS
Program Name:	_____	_____	_____
Version No.:	_____	_____	_____
Vendor:	_____	_____	_____

D-514 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

5. SUBSTRUCTURE APPURTENANCES**Protection Systems****ABUTMENTS****PIERS**

Corrosion: _____

Collision: _____

End Treatment

Railing/Fencing: _____

Barrier: _____

Special Backfill/Grading Requirements:

Structural Backfill: _____

Slope Protection: _____

Slope Benching: _____

Slope Grading: _____

Other: _____

Scour Protection

Abutments: _____

Piers: _____

Substructure Drainage:

Abutments: _____

Piers: _____

6. COMMENTS

D-515 (5-12)



QUALITY ASSURANCE FORM FOR INTEGRAL ABUTMENT BRIDGES

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Skew: _____ Grade: _____

Bridge Type: _____

Design ADT: _____ Year: _____ ADTT: _____ Year: _____

No. of Spans and Span Lengths: _____

2. PILE DESIGN

Pile Type/Size: _____; No. of Piles/Beam _____

Total expansion length/thermal movement:

Abutment 1 _____; Abutment 2 _____; Calc. Page _____

Estimated Pile Length: Abutment 1 _____; Abutment 2 _____

Pile Loads: Vertical Resistance _____ Design _____

Lateral Resistance _____ Design _____

Interaction Ratio for Axial + Bending (A6.9.2.2) _____

Pile Orientation: Abutment 1 _____; Abutment 2 _____

Assumed Depth to Fixity: Abutment 1 _____; Abutment 2 _____

Actual Depth of Fixity: Abutment 1 _____; Abutment 2 _____

Note: The above point of fixity information is to be taken from the COM624P or LPILE analysis as specified in Section D6.15.3.3 of DM-4.

D-515 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

3. INTEGRAL ABUTMENT DESIGN

Integral Abutment: Full ☐ Partial ☐

Lateral loads due to passive earth pressure: _____

Has stability of beam erection been checked? Yes ☐ No ☐

Backwall cushion material type/thickness: _____

Has subsurface drainage been addressed? Yes ☐ No ☐

Has provision been made for approach slab sliding? Yes ☐ No ☐

Wingwall Design: Independent ☐ Moving with Bridge ☐

Expansion Dam: Type _____; Movement Class _____

Other special design details: _____

4. MULTI-SPAN STRUCTURES

(QA Form D-514 will apply and must be completed)

	Pier 1	Pier 2	Pier 3
Foundation Type:	_____	_____	_____
Support Type:	_____	_____	_____
Design Height:	_____	_____	_____
Bearing Fixity:	_____/____	_____/____	_____/____

5. COMMENTS

D-516 (5-12)



QUALITY ASSURANCE FORM FOR FINAL DESIGN OF PROPRIETARY RETAINING WALLS

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Over: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Design ADT: _____ ADTT: _____ Year: _____

2. PROPRIETARY WALL INFORMATION

Wall Type: RE ☐ Other ☐, indicate other _____

Application: Retaining Wall ☐ Abutment ☐

Length: _____ ft.; Max. Height _____ ft.; Design height _____ ft.

Traffic Surcharge = _____ ft.

Fill Slope = _____, Height = _____ ft.

3. TECHNICAL INFORMATION

(a) Stability Criteria (External)

Eccentricity (on soil) = _____ < 3B/8

Eccentricity (on rock) = _____ < B/4

Slope Stability (if req'd) = _____ > 1.5

Maximum Horizontal Loads _____; Sliding Resistance = _____

(b) Max. Foundation Pressure (ksf): Design = _____; Bearing Resistance = _____

(c) Stability (Internal)

Pull out force = _____; Ultimate Pull Out Capacity = _____

Depth Level = _____ ft.

(d) Estimated Settlement (Max.) _____ in.

(e) Least ratio of soil reinforcement length to wall height _____;

Location _____

D-516 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

(f) Backfill:

REINFORCED ZONE

RANDOM

Drained Angle of Internal Friction

 $\phi =$ _____ °; $\phi =$ _____ °;

Total Density of Material

 $\gamma =$ _____ kcf $\gamma =$ _____ kcf**FOR WALL APPLICATION COMPLETE SECTION 4 ONLY****FOR ABUTMENT APPLICATION COMPLETE SECTIONS 4 AND 5****4. WALL APPLICATION**

(a) Panel Thickness _____ in.; Minimum Cover for Rebars _____ in.

(b) Indicate any deviations from standard construction specifications: _____

(c) Drainage Requirements:

Indicate design assumptions for drainage of backfill material, on top and under the wall: _____

Is drainage provided to validate design assumptions: Yes ☐ No ☐**5. ABUTMENT APPLICATION**(a) Pile Type: Point Bearing ☐ Friction ☐ End Bearing ☐

(b) Pile Size: Steel _____; Concrete _____

(c) Actual Max. Load/Pile _____ tons; Allowable Load/Pile _____ tons

(d) Type of arrangement (Load Transfer Mechanism) of soil-reinforcing elements around piles: _____

D-516 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

6. COMMENTS

D-517 (5-12)



QUALITY ASSURANCE FORM FOR FINAL DESIGN OF FLEXIBLE RETAINING WALLS

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Along: _____
(STREAM, RAILROAD, OR ROAD)

S-No.: _____ Stations: Begin Wall _____; End Wall _____

2. GEOMETRIC DESIGN INFORMATION

Wall Type: Permanent ☐ or Temporary ☐

Cantilever ☐ or Tie Back/Anchored ☐

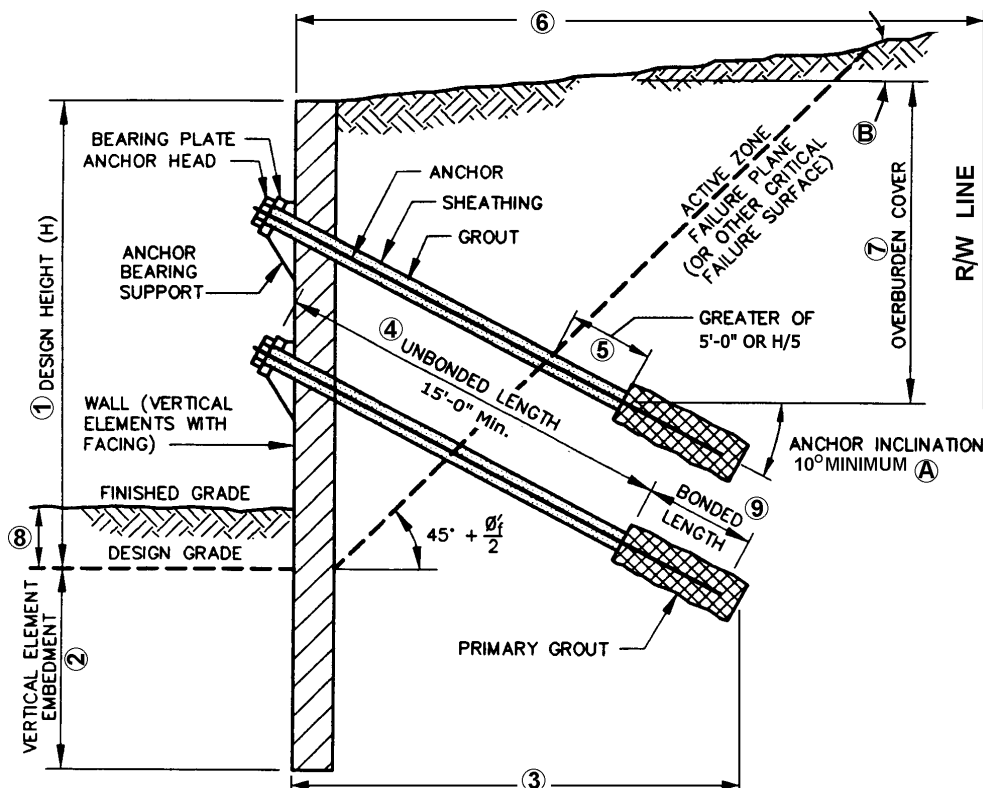
If Tie Back/Anchored Wall, No. of Anchors per Vertical Element: _____

Permanent Tie Back/Anchored Walls:

Method of Installation: Top Down ☐ or Bottom Up ☐

Lagging Type: Precast ☐ ; Timber ☐ or N / A ☐

Support/Pile Type: Double Pile ☐ ; Single Pile ☐ or Internal ☐



D-517 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Design

Dimensions →

1 = _____ ft.; Calc. Page _____

2 = _____ ft.; Calc. Page _____

3 = _____ ft.; Calc. Page _____

4 = _____ ft.; Calc. Page _____

5 = _____ ft.; Calc. Page _____

6 = _____ ft.; Calc. Page _____

7 = _____ ft.; Calc. Page _____

8 = _____ ft.; Calc. Page _____

9 = _____ ft.; Calc. Page _____

Angles →

A = _____ Degrees; Calc. Page _____

B = _____ Degrees; Calc. Page _____

3. SOIL AND FOUNDATION DATA

In-Situ Soil Type: _____ ; Calc. Page _____

Unit Density = _____ kcf; Cohesion (c) = _____ kcf; Calc. Page _____

Angle of Internal Friction = _____ ; Calc. Page _____

Stability Number (N) (D3.11.5.7) = _____ ≤ 3.0; Calc. Page _____

Foundation Material:

Soil ☐ Rock ☐ ; Calc. Page _____

Type _____ ; Calc. Page _____

Unit Density = _____ kcf; Cohesion (c) = _____ kcf; Calc. Page _____

Angle of Internal Friction = _____ ; Calc. Page _____

Stability Number (N) (D3.11.5.7) = _____ ≤ 3.0; Calc. Page _____

Foundation Design:

Pressure: Resistance = _____ ; Actual = _____ ; Calc. Page _____

Settlement: Allowable = _____ ; Actual = _____ ; Calc. Page _____

D-517 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Slope Stability Analysis Performed? Yes ☐ No ☐ ; Calc. Page _____

Bearing Resistance = _____ ; Calc. Page _____

Maximum Bearing Pressure = _____ ; Calc. Page _____

Live Load Surcharge Used = _____ ; Calc. Page _____

Earth Pressure Used: Active ☐ Passive ☐ ; Calc. Page _____

4. VERTICAL ELEMENT DATA

Type: _____ ; Size: _____ ; Calc. Page _____

Spacing: _____ ; Embedment Length = _____ ; Calc. Page _____

Design Checked for Staged Construction? Yes ☐ No ☐ ; Calc. Page _____

Corrosion Protection _____

Special Details: _____

5. ANCHOR DATA

Type: _____ ; Calc. Page _____

Size: _____ ; Spacing: _____ ; Calc. Page _____

Loads: Resistance = _____ ; Design: _____ ; Calc. Page _____

Bond Length based on: Soil ☐ Rock ☐ ; Calc. Page _____

Corrosion Protection _____

Is Anchor Installation Procedure included? Yes ☐ No ☐

6. LAGGING AND FACING DATA

Lagging: Temporary ☐ Permanent ☐ ; Calc. Page _____

Type: _____ ; Calc. Page _____

Maximum Design Bending Moment = _____ ; Calc. Page _____

D-517 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Facing: Cast-in-Place Concrete ☐ Precast Concrete ☐ ; Calc. Page _____

Concrete Class = _____; Thickness = _____ ; Calc. Page _____

Reinforcement: Wire Mesh ☐ or Bars ☐ ; Calc. Page _____

Plain ☐ or Epoxy-Coated ☐ ; Calc. Page _____

Maximum Design Bending Moment = _____ ; Calc. Page _____

Attachment Details Designed? Yes ☐ No ☐ ; Calc. Page _____

7. DRAINAGE DETAIL DATA

Are Drainage Panels provided behind wall? Yes ☐ No ☐

Do Drainage Panels extend full height of wall? Yes ☐ No ☐

Is Insulation provided to prevent freeze/thaw damage Yes ☐ No ☐

8. MISCELLANEOUS DATA

If Tie Back/Anchored Wall is in a fill situation, is the necessary approval from the

Chief Bridge Engineer included with the submission? Yes ☐ No ☐

For Tie Back/Anchored Walls, do the plans and special provisions for this submission contain the proof,

performance, creep, and lift off testing of the anchors? Yes ☐ No ☐

9. COMMENTS

D-518 (5-12)



QUALITY ASSURANCE FORM FOR FINAL DESIGN OF SOUND BARRIERS

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

1. PROJECT INFORMATION

Project MPMS/ECMS No.: _____

County: _____ S.R.: _____ Sec.: _____ Along/Over: _____
(ROADWAY, RAMP, ETC.)

S-No.: _____ Stations: Begin Wall _____; End Wall _____

2. SOUND BARRIER WALL - GENERAL

Ground-Mounted: Post & Panel ☐ Offset ☐

Structure-Mounted: Bridge-Mounted ☐ Moment Slab-Mounted ☐ Retaining Wall-Mounted ☐

If Bridge-Mounted, Bridge Inspectability Provided? Yes ☐ No ☐

Total Barrier Length: _____ ft.

Design Build? Yes ☐ No ☐

If Design Build, Foundation Parameters Provided by Designer? Yes ☐ No ☐

If Design Build, Foundation Parameters Provided by Contractor? Yes ☐ No ☐

Block and Note for District Environmental Manager Signature? Yes ☐ No ☐

Acoustic Profile Indicated? Yes ☐ No ☐

Antigraffiti Coating? Yes ☐ No ☐

If Yes: Limits Indicated: Yes ☐ No ☐

Federal Color Number: _____

Penetrating Concrete Stain? Yes ☐ No ☐

If Yes: Limits Indicated: Yes ☐ No ☐

Federal Color Number: _____

Integral Color Pigmentation for Precast Concrete? Yes ☐ No ☐

If Yes: Federal Color Number: _____

D-518 (5-12)

Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

3. POSTS

Design done by? Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No. _____

Post Type: Steel ☐ Precast Concrete ☐ Other ☐, indicate other _____

Post Connection: Baseplate ☐ Embedded ☐ Other ☐, indicate other _____

Is Connection Patented? Yes ☐ No ☐

Design Wind Pressure: _____ psf

Design Post Spacing: Minimum _____ ft.; Maximum: _____ ft.

Design Wall Height: Minimum _____ ft.; Maximum: _____ ft.

If Concrete Post: Reinforcement? Uncoated ☐ Epoxy-Coated ☐ Galvanized ☐

If Steel Post: Federal Paint Color Number: _____

4. PANELS

Design done by? Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No.: _____

Panel Material: Concrete ☐ Steel ☐ Other ☐, indicate other _____

Sound Absorptive Material Required? Yes ☐ No ☐ If Yes, indicate PE No.: _____

Access Door Required and Detailed? Yes ☐ No ☐

Sleeve Openings Required and Detailed? Yes ☐ No ☐

Steel Cables thru Panels and Connected to Steel Posts? Yes ☐ No ☐

Design Wind Pressure: _____ psf

Design Post Spacing: Minimum _____ ft.; Maximum: _____ ft.

Design Panel Height: Minimum _____ ft.; Maximum: _____ ft.

Structural Panel Thickness: _____ inch

Architectural Surface Treatment:

Roadway Side: Surface Treatment: _____ Average Thickness: _____ in.

Residential Side: Surface Treatment: _____ Average Thickness: _____ in.

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Designer: _____ Date: _____
 (DESIGN OFFICE & NAME OF DESIGNER)

Total Average Panel Thickness: _____ in.

Minimum Number of Lifting Inserts Indicated? Yes ☐ No ☐

Reinforcement? Uncoated ☐ Epoxy-Coated ☐ Galvanized ☐

5. FOUNDATIONS (for Ground-Mounted Walls only)

Design done by?

Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No.: _____

Site Specific Design ☐ If Site Specific Design, Specify why? _____

Type: Spread Footing ☐ Drilled Caisson ☐ Other, Specify: _____

Ground Surface: Level ☐ Sloped ☐ If Sloped, Specify slope: _____

Spread Footings:

Design Wind Pressure: _____ psf

Maximum Foundation Pressure (ksf): Allowable _____; Design _____

Coefficient of Sliding Friction: _____

Max. Dimensions: W _____; L _____; Thickness _____

Max. Estimated Settlement _____ in.

Overexcavation below bottom of footing required? Yes ☐ No ☐

If required, Indicated backfill material type: _____

Drilled Caissons:

Design Wind Pressure: _____ psf

Soil Type (if per Standards): _____

Soil Properties (Site Specific):

Angle of Internal Friction: _____ Cohesion psf: _____

Unit Weight pcf: _____ Modulus of Subgrade Reaction pci: _____

e50: _____

Design Size: Maximum Diameter _____; Maximum Length _____

D-518 (5-12)

Designer: _____ Date: _____
(DESIGN OFFICE & NAME OF DESIGNER)

6. OFFSET GROUND MOUNTED WALLS

Design done by? Standards ☐ New Product Evaluation (PE) ☐ If PE, indicate PE No.: _____

Cables Connections:

Minimum of two cable connections per panel-to-panel connection? Yes ☐ No ☐

Minimum of three cable connections for end panel to adjacent panel(s) connection? Yes ☐ No ☐

7. GENERAL

Indicate any deviations from standard design, new product evaluation (PE), construction, or material specifications _____

8. COMMENTS

D-519 (5-12)



QUALITY ASSURANCE FORM FOR CONSTRUCTION LOAD ANALYSIS

FOR DEPARTMENT USE ONLY

Designer: _____ Q/A Reviewer: _____ Review Date: _____

County: _____ S.R.: _____ Segment: _____ Offset: _____

Contractor: _____ ECMS No.: _____ Structure Type: _____

(Check Applicable Box)

Yes No N/A

1. SUBMISSION REQUIREMENTS

Are submitted materials signed and sealed by PE registered in PA? ☐ Yes ☐ No ☐ N/A

Letter of submission for loadings:

Construction loads comply with Section 105.17 of Pub. 408? ☐ Yes ☐ No ☐ N/A

100 kips maximum per span in 12-foot construction width? ☐ Yes ☐ No ☐ N/A

200 kips maximum per span? ☐ Yes ☐ No ☐ N/A

Are construction loads reduced due to weight restricted structure? ☐ Yes ☐ No ☐ N/A

Does the submission identify type of construction (Scope of Work)?

Painting? ☐ Yes ☐ No ☐ N/A

Milling/Overlay? ☐ Yes ☐ No ☐ N/A

Crane operating on bridge? ☐ Yes ☐ No ☐ N/A

Is construction phased? ☐ Yes ☐ No ☐ N/A

Are traffic control plans provided? ☐ Yes ☐ No ☐ N/A

Are lane locations/restrictions identified? ☐ Yes ☐ No ☐ N/A

Are separate models required for phased construction? ☐ Yes ☐ No ☐ N/A

Are construction loads and locations clearly identified? ☐ Yes ☐ No ☐ N/A

District's approval letter and stipulations to Contractor documented? ☐ Yes ☐ No ☐ N/A

District Rating Engineer notified bridge under construction? ☐ Yes ☐ No ☐ N/A

Has the District Rating Engineer acknowledged the construction loads? ☐ Yes ☐ No ☐ N/A

Has the District updated the APRAS file? ☐ Yes ☐ No ☐ N/A

Is updated APRAS file linked to bridge? ☐ Yes ☐ No ☐ N/A

Were Engineering Dataset(s) submitted? ☐ Yes ☐ No ☐ N/A

Are separate dataset(s) provided for phased construction? ☐ Yes ☐ No ☐ N/A

2. ANALYSIS REQUIREMENTS

Is analysis signed and sealed by PE registered in PA? ☐ Yes ☐ No ☐ N/A

Was PennDOT Approved Software (DM-4, Appendix J) utilized? ☐ Yes ☐ No ☐ N/A

Hand calculations reflect appropriate analysis approach, formulas, solutions? ☐ Yes ☐ No ☐ N/A

D-519 (5-12)

Designer: _____ Q/A Reviewer: _____ Review Date: _____

	(Check Applicable Box)		
	Yes	No	N/A
Method of analysis:			
ASD rating analysis method used?	☐	☐	☐
LFD rating analysis method used?	☐	☐	☐
LRFD rating analysis method used?	☐	☐	☐
Load factors used for construction loads per DM4/AASHTO?	☐	☐	☐
Controlling DFm: _____			
Controlling DFv: _____			
Dead Loads:			
Are stock piles and construction equipment included?	☐	☐	☐
Are additional temporary barriers included?	☐	☐	☐
Are curbs, appurtenances, falsework attachments, etc. included?	☐	☐	☐
Live Loads:			
Are construction live loads included (milling operation, etc.)?	☐	☐	☐
Are "Design / Permit" vehicle live loads included?	☐	☐	☐
Are appropriate lanes loaded with live traffic?	☐	☐	☐
Are the number of lanes and lane widths correct?	☐	☐	☐
Are the appropriate lanes loaded with permit vehicle?	☐	☐	☐
Number of Lanes Loaded _____			
Are the applicable lanes loaded with constructions loads?	☐	☐	☐
Number of Lanes Loaded _____			
Have all legal loads been analyzed?	☐	☐	☐
Are traffic restrictions required and identified?	☐	☐	☐
Is the live load impact (I) or dynamic load allowance (IM) factor correct?	☐	☐	☐
Are Sidewalks and Pedestrian load considered?	☐	☐	☐
Has the most recent inspection documentation been referenced and considered?	☐	☐	☐
Is "Section Loss" present and included in the analysis?	☐	☐	☐
Are separate models provided for phased/partial construction condition?	☐	☐	☐
Do Distribution Factors (DF) reflect phased/partial construction condition?	☐	☐	☐
Are Construction Load Capacity envelopes provided?	☐	☐	☐

D-519 (5-12)

Designer: _____ Q/A Reviewer: _____ Review Date: _____

	(Check Applicable Box)		
	Yes	No	N/A
Are crane loadings present and included?	☐	☐	☐
Are special live load and axle loading correct?	☐	☐	☐
Is false work analysis included and adequate loads used?	☐	☐	☐
Are partial demolition effects considered?	☐	☐	☐
Span length match drawing data or field measurements?	☐	☐	☐
Is a horizontally curved girder analysis required and provided?	☐	☐	☐
Splayed girders present and considered in distribution factor?	☐	☐	☐
Is top flange bracing considered appropriately?	☐	☐	☐
Are transverse stiffeners present and included?	☐	☐	☐
Are flange plate and web dimensions correct?	☐	☐	☐
Are P/S girder or box beam dimensions correct?	☐	☐	☐
Does strand pattern (cgs data) match plans?	☐	☐	☐
Are truss members and dimensions and geometry correct?	☐	☐	☐
Are material properties correct?	☐	☐	☐
Are member properties correct?	☐	☐	☐
Correct diaphragm spacing used?	☐	☐	☐
Correct girder spacing used?	☐	☐	☐
Non-composite deck construction considered?	☐	☐	☐
Composite deck construction considered?	☐	☐	☐
Continuity considered appropriately?	☐	☐	☐
Skew applied?	☐	☐	☐

3. CRITICAL MEMBER CAPACITIES

Moment:

Controlling Member: _____ Location: _____

Maximum Capacity (Kip-Ft) _____ Maximum Loading (Kip-Ft) _____ Operating Rating Factor _____

Shear:

Controlling Member: _____ Location: _____

Maximum Capacity (Kips) _____ Maximum Loading (Kips) _____ Operating Rating Factor _____

D-519 (5-12)

Designer: _____ Q/A Reviewer: _____ Review Date: _____

4. REMARKS

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX B – TECHNICAL GUIDELINES FOR SHOP DRAWING REVIEW

1. The following is provided to emphasize compliance with PP1.10.2.5, Publication 408, Section 105.02(d) and promote a consistent and efficient preparation, review and processing of bridge shop drawings and working drawings, when required.

(a) Drawings

Drawings created electronically using a CAD program are preferred and encouraged. When the drawings are created using CAD, the PDF file is to be generated directly from the CAD software and not via a scanning process of a printed hardcopy. If the drawings are created manually, the review and further processing are to proceed electronically. The Contractor is responsible for transmitting a PDF file created by scanning the hardcopy originals. The scan resolution is to be 300 dpi in 'black-and-white' mode.

The composition of shop drawings is to conform to the following:

- Sheet size: 22"x34" (ANSI 'D')
- Margins: 1.5" left side, 0.5" right side and top and bottom
- Minimum text: 0.10" height (Leroy equivalent L100) *Note this is a deviation from Pub. 408 Sec. 105.02(d), which stipulates L120 as the minimum text size, because of the use of CAD to generate drawings*
- Text width compression: may be used on limited basis at the sole discretion of the reviewer or Department

A title block shall be placed in lower right-hand corner or along the bottom margin legible while viewed in landscape mode. A title block oriented along the right side margin legible when viewed in portrait mode is unacceptable. The title block must contain the following information as a minimum:

- County
- Route
- Section number
- Segment and offset
- ECMS contract number
- Name of Contractor
- Name of Fabricator
- Drawing title
- Drawing number
- Structure number (S-number)
- Initials of the drawer
- Initials of the checker
- Date of initial drawing creation

A review block 2" high minimum (4" per 105.02(d)) x 3" wide for the review stamp immediately adjacent to the title block (preferably to the immediate left.)

The shop drawings shall to be stamped "Returned for Correction" if the above requirements are not included.

Partial shop drawing submissions are discouraged, but may be permitted on a case-by-case basis only with the approval of the District Bridge Engineer. A typical example of when the use of partial submissions may be justified involves material supplied in different construction seasons because of phased construction.

For a sample shop drawing sheet conforming to the above requirements, see Figure 1-1.

The above criteria are not mandatory for working drawings (Publication 408, Section 105.02(c)), but are encouraged for consistency.

(b) Review Stamps

The only permitted stamps in ascending level of status are "Returned for Corrections", "Accepted as Noted" and "Accepted". The entire plan set is to be stamped with the same stamp controlled by the lowest status level. For example, one sheet cannot to be stamped "Accepted", with another sheet stamped "Accepted as Noted", while another sheet is stamped "Returned for Corrections". If the review status of just one sheet in the set is "Returned for Corrections", then all sheets are to be stamped "Returned for Corrections".

Use electronic stamps generated in Adobe Acrobat 'Professional' version, or equivalent software, so that the entire drawing review process can be executed electronically without having to scan manual stamps placed on hardcopy prints and subsequently scanned. The stamp is to contain the full name of the person responsible for the review, the reviewer's title and the reviewer's signature (not just the reviewer's initials) along with the date.

The only qualifying language permitted to appear with an "Accepted" or "Accepted as Noted" stamp is the following statement *"Acceptance does not relieve the Contractor from their responsibility for full conformance to the contract documents."*

The following are samples of stamps to be used.

PennDOT Engineering District _-0 ACCEPTED
Date
{Reviewer's Signature}
"Insert Full Name of Reviewer" "Insert Title of Reviewer"

PennDOT Engineering District _-0 ACCEPTED AS NOTED
Date
{Reviewer's Signature}
"Insert Full Name of Reviewer" "Insert Title of Reviewer"

PennDOT Engineering District _-0 RETURNED for CORRECTIONS
Date
{Reviewer's Signature}
"Insert Full Name of Reviewer" "Insert Title of Reviewer"

(c) Review Mark-ups

Mark-up electronic of shop drawings using Adobe Acrobat 'Professional' version, or equivalent software, annotation tools is preferred and encouraged in order to improve legibility while reducing file size. Annotation and commenting features are available in the free 'Reader' version of Acrobat, but it will be the reviewer's responsibility to convert the source PDF file

into the 'Commenting Enabled in Reader' version. The Department's preference is for the consultant reviewer to have access to the 'Professional' version of Acrobat.

If a contractor generates PDF files directly from a CAD program and the consultant reviewer chooses to manually mark-up the drawings, it is the consultant's responsibility to produce the hardcopy prints at their expense. Manually marked-up review prints may be scanned as a last resort. Use 'color' scan mode at 200 dpi resolution. The final "ACCEPTED" shop drawing set of record must be stamped electronically.

(d) File Naming Convention

The submission from the Contractor or Fabricator to the Reviewer shall have the following file name convention:

<ECMS No.>_<Date>_<S-number>_SHD_<Description>_(submission number).PDF

- Use the following format for the date field: YYYY-MM-DD
- The description is to be kept short and concise for example, "Struc_Steel", "Bearings", "Strip_Seal_Dams", "PS_Beam", "PC_Box_Culvert", "SIP_Forms"

The following is an example file name for the second submission of prestressed concrete bridge beams:

21782_2009-12-11_S-28839_SHD_PS Beams_(2).PDF

Use "WKD" as the three character code for working drawings. Example working drawings include: Beam Erection, Structure Demolition, Superstructure Jacking Plans and Procedures, Painting Containment, Deck Overhang Formwork, Temporary Excavation Support and Protection Systems, Work Platforms, etc..

The following is an example file name for a working drawing submission:

21782_2009-12-11_S-28839_WKD_TESPS_(1).PDF

The response submission from the Reviewer to the Contractor or Fabricator shall modify the submission number designation with one of the following annotations based on review status and append the date of the reply in YYYY-MM-DD format.

- (_-A) = Accepted
- (_-AAN) = Accepted as Noted
- (_-RFC) = Returned for Corrections

The following is an example file name for a Reviewer's response submission stamped "Returned for Corrections":

21782_2009-12-11_S-28839_SHD_PS Beams_(2-RFC)_2009-12-24.PDF

(e) Miscellaneous File Details

Once the shop drawings are stamped in Adobe Acrobat, the shop drawings are to be re-printed to the Adobe printer (select Adobe PDF as the printer name) and a new PDF file generated so that the stamp is embedded within the file and cannot be subsequently modified. Do NOT submit a digitally 'signed' or 'certified' document or apply any other type of security setting.

(f) Submission and Distribution

Electronic submission and distribution of shop drawings and working drawings is preferred. Coordinate with the District on the preferred method for transmitting shop drawings and working drawings (i.e. e-mail, ftp site, hard copy, etc.).

The recommended method for submissions is via a secure internet site. The Contractor shall create a secure internet site for the submission of shop drawings and working drawings. Provide a username and password for access to the site. Include in the site a project folder identified with the project's ECMS number, the structure S-number, and the District's bridge name.

Create individual subfolders for each category of shop drawing and working drawing (“Struc. Steel”, “Bearings”, “Strip Seal Dams”, “Falsework”, etc.) and match the subfolder name with the category of shop drawing or working drawing. Place all “Accepted” shop drawings in a subfolder entitled “Accepted Shop Drawings”. The internet site shall be maintained until 90 days past the last day of physical work.

The Contractor or Fabricator shall send out an email when shop drawings or working drawings are posted on the secure internet site. The subject line in the distribution e-mail must include the ECMS project number, the structure S-number, a short description of the shop drawing or working drawings along with the current submission number.

A formal Transmittal Letter is required for all shop drawing submissions, whether from the Contractor or Fabricator to the Reviewer or from the Reviewer to the Contractor or Fabricator. A distribution e-mail without a formal transmittal letter attached is unacceptable.

Distribution list for all distributions (including “Returned for Corrections”):

- Fabricator
- Contractor
- District Project Engineer
- District Bridge Engineer or Designee
- District Structure Control Engineer

Distribution list for the distribution of “Accepted” and “Accepted as Noted” drawings:

- All parties indicated above, plus the following additional recipients:
- PennDOT shop inspector (sent c/o Fabricator), when required
- PennDOT Bridge Design and Technology Division, ATTN: Structural Materials Section, when required
- PennDOT ACE (Assistant Construction Engineer)
- PennDOT Inspector-in-Charge (name supplied at pre-construction meeting)

Electronic files of a shop drawing submission in PDF format are acceptable for all parties, except the PennDOT shop inspector and the PennDOT inspector-in-charge, who require full-size hardcopy sheets of the “Accepted as Noted” and “Accepted” sets. An acceptable practice is to provide the fabricator with an electronically stamped PDF file, who, in turn, will plot out a full-size hardcopy set for the shop inspector. The contractor is responsible for supplying the shop inspector with a full-size hard copy set if the fabricator does not have this capability. The Contractor shall supply the inspector-in-charge with a full-size hardcopy set of the drawings.

The Department’s Structural Materials Section (SMS) will accept electronic copies of submissions. Only shop drawings requiring plant inspection need to be sent to the SMS. For example, shop drawings for SIP deck forms are not to be sent to the SMS, since no plant inspection is required. Working drawings are not to be sent to the SMS. Plant inspection is required for, but not limited to, products defined in Publication 408, Section 601.2(a)3.a - Special Design Concrete Pipe, Section 714 - Precast Concrete Products, Section 1085 - Precast Reinforced Concrete Box Culverts, Section 1086 - Sound Barriers, Section 1101 - Highway Lighting Poles, Section 1104 - Traffic Signal Poles, Section 1105 - Fabricated Structural Steel and Aluminum and Section 1107 - Prestressed Concrete Bridge Beams.

Copy the “Accepted” or “Accepted as Noted” shop drawings to the following folder on the Department’s ftp site:

ftp://ftp.dot.state.pa.us/transfer/MTD_Structural_Shop_Dwgs_Inbox

Please ensure that proper file naming conventions as defined in (d) above are followed. Send an e-mail notification to RA-pdSTRUCTMATLS@state.pa.us indicating the shop drawings have been copied to the ftp site.

All files for District personnel are to be copied to the appropriate District folder on the Department’s ftp site:

<ftp://ftp.dot.state.pa.us/transfer>

This is an unsecured folder and a log-in procedure is not required. An e-mail notification with an accompanying transmittal letter is required to be sent to the distribution parties listed above. Each District may specify a different location/folder on the ftp site based on the project or District preferences.

(g) Release of Materials from Fabricator for Shipment to Project

Shop drawing sets stamped “Accepted as Noted” permit the fabrication process to commence, but the material is not to be released for shipment until an “Accepted” set of shop drawings has been issued and distributed. Only a shop drawing set stamped “ACCEPTED” will serve as the official set of record.

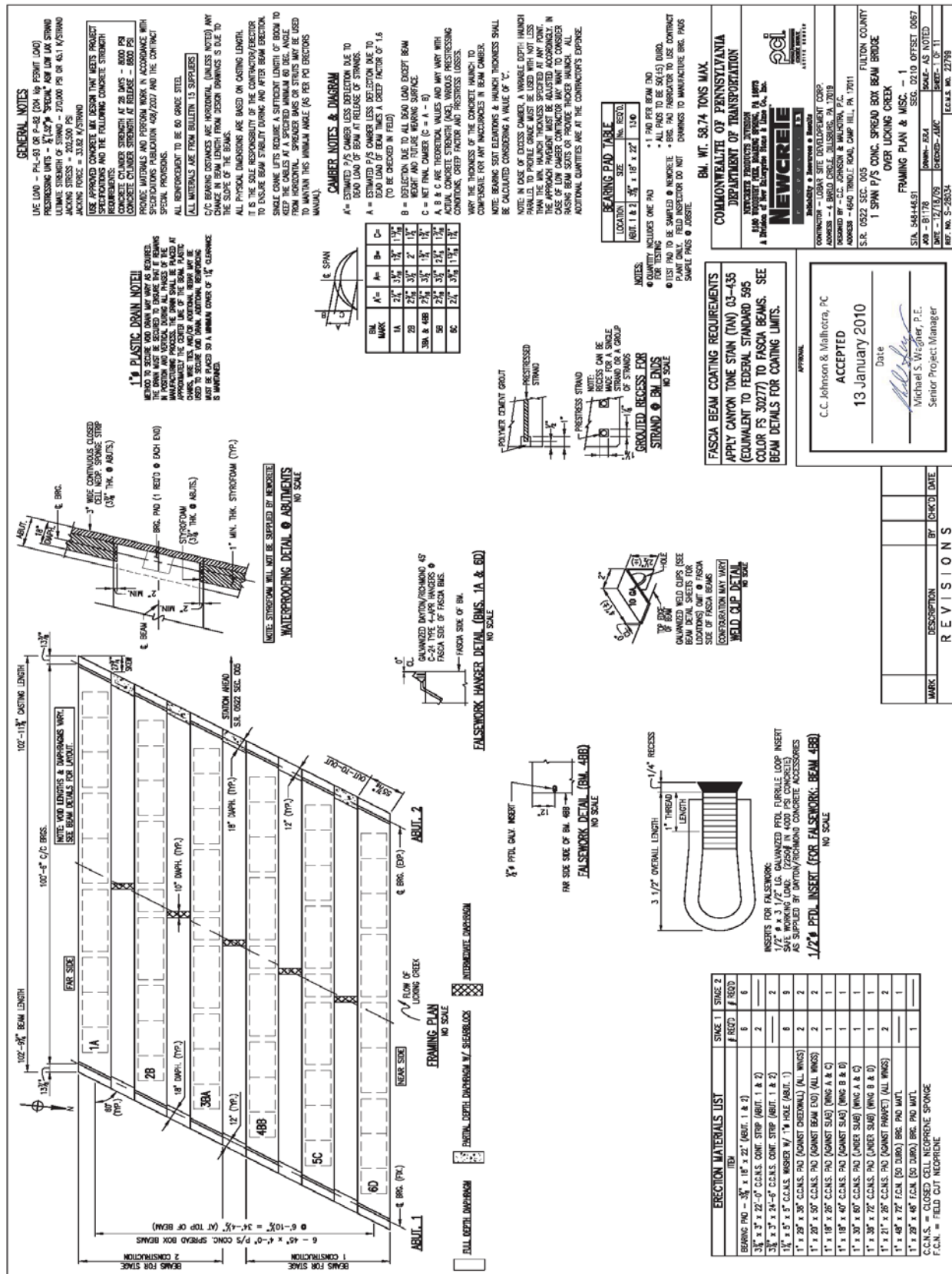


Figure 1-1 – Sample Shop Drawing

2. The following contains technical guidelines for the review of shop drawings based on the item(s) submitted. For Policy and Procedure related to shop drawing review see PP1.10.2. Use the attached standard transmittal letters as appropriate.

(a) Fabricated Structural Steel

Check the Fabricator's geometrics for conformance to principal dimensions shown on the contract plans. This review does not include the check of detail dimensions.

Check all materials for conformance with the requirements of the contract plans and specifications. This includes painting requirements and material testing requirements.

Check that beams and girders are detailed to provide the camber as required by the contract plans.

Check that the profile of the girders are consistent with the vertical shape of the bridge.

The direction of rolling for splice plates or other small plates which may be easily disoriented should be shown on the shop drawings (see Publication 408, Section 1105.03(s)).

Check all splices, joints and connections for conformance with the intent of the contract plans and specifications. This check shall include conformance of weld types and sizes.

Check all web plates for thickness, including splice locations, and depths, including tapers and end haunches.

Check flange plates for thickness, including splice locations, width, and conformance to minimum length requirements.

Check stiffeners and connection plates for width, thickness, welding, and approximate spacing of intermediate stiffeners if any are required.

Check all bolted field splices for size of splice material and number and size of bolts in bolted material.

Check all cross frame connections for number of bolts in connection plate, length and size of welds, and size and type of members.

Check center of span, field splice, and interior pier ordinates.

Check fillet weld sizes for all weld connections for web to flange, stiffeners to web and flange, and built-up members.

Check number and size of bolts in floor beam, diaphragm, and cross girder connections.

For fracture-critical members, be sure that all necessary information on the design drawings and in the fracture control plan is properly shown on the shop drawings.

Nondestructive testing required for each welded joint type should be listed or identified on shop drawings to inform the shop inspector of appropriate testing and location.

Check that all details follow the latest revision to contract plans.

(b) Grid Floor

Materials furnished must be in accordance with the designer's selection from the manufacturer's catalog as stated in the Special Provisions of the contract. Moment of inertia, section modulus and density of the grid panels must be as required per design. The welding process and electrodes used must be shown. Full bearing at 90% of the grid intersections is required, and all intersections must be welded. These requirements should be noted on the shop drawings.

(c) Expansion and Fixed Dams

Ensure that clip angles (or plates) used to support plate and fingerdams on the stringers and diaphragms are of sufficient quality and spacing, and of sufficient depth and correct vertical alignment to permit attachment to pre-drilled holes, inserts or attachment plates in the supporting members.

If adjustable support clips or plates are used, the assemblies must be welded after adjustment; this requirement must be shown on the shop drawings.

On shop drawings for armored neoprene compression seals, the location of the lug bars should be about 1/2 in. or more below the compressed height as tested by the Laboratory Testing Section of the Innovation and Support Services Division.

Shop drawings for modular expansion dams, components of which are by themselves structural members (continuous beams), should show all shop splices, if any.

Splices must be made with full penetration welds or partial penetration welds augmented by improved section properties at the splices, if/or as approved by the Bridge Design and Technology Division.

A "Temperature-Joint Opening" chart ranging from -10° F to 100° F in 10° increments must be shown on the shop drawings for neoprene compression seals.

Ensure that the selected seal will be at least 20% compressed at full opening of the joint. Full opening consists of opening at construction temperature, effect of dead load rotation of bearings at the joint, and construction of the superstructure at lowest erection temperature.

The welding process and the electrodes must be shown on the shop drawings.

Special attention must be given to proprietary expansion dams, particularly the support systems. Approved proprietary dams will be listed in Bulletin 15, but such approval is intended primarily for the leakproof and movement capabilities of the dam. The support system for proprietary expansion dams must be equivalent to the support system shown in the Department's expansion dam standard drawings, unless otherwise detailed on the design drawings. When in doubt, contact the Bridge Design and Technology Division for guidance.

(d) Railings and Barrier Protective Fences

There should be 9 in. minimum distance between the centerlines of the railing post and adjacent deflection joints, this should be checked at the time of shop drawing review. Also check that railings are spliced at expansion joints with articulated splices.

(e) Drainage Items

Ascertain that fabricator uses correct piping details at expansion joints. Bolt holes in scupper bases must be compatible with bolt holes in supporting diaphragms or stringers.

(f) Bearings

Check that surface finishes on matching bearing plates are specified. If bearing assemblies require welding on plates which have specified finishes, the finishing should be done after welding is complete. This will assure surface flatness and eliminate the effects of warpage, if any, due to welding.

Check the orientation of the bearings, both relative to the girder as well as to bearing components.

Check that the materials, surface finishes and details for pot bearings are in conformance with contract plans.

(g) Sign Structures

Check for correct member sizes and welding and bolting details. Make sure hand holes are provided in sign structures that are illuminated. Weep holes must be provided at bottom of tower shafts. Ensure that needed camber for trusses is shown. Camber should be provided as per applicable Standard Drawings BD-642M, BD-643M, BD-644M, BD-645M and BD-647M.

(h) Light Poles

Check shop drawings and structural computations for structural capacity only. Electrical details are to be reviewed and approved by the District or Bureau of Maintenance and Operations, Maintenance Technical Leadership Division. Check for prior light pole approval by light pole manufacture under a General Submission.

(i) Pre-Tensioned and Post-Tensioned Concrete Beams and Panels

The shop drawings must show a framing plan for the whole structure, including proper beam identification for each beam. The force and eccentricity for all beams must conform to the design drawings within reasonable tolerance. Major deviations must be substantiated by calculations submitted by the fabricator with the shop drawings. Check the beam lengths and continuity details against the design assumptions and construction plan.

Concrete release strength and 28-day strength must be shown on the shop drawings, as well as strand patterns and all cast-in hardware, voids or other components. Generally, the Department accommodates prestress fabricators and allows deviations from design plans if allowable stresses are not exceeded and the Department receives a product as good as or better than designed, at no extra cost.

Check that tensile stresses in the top fiber at centerline of bearings of box beams are within allowable stresses, or have been reduced to allowable stresses by either unbonding and/or unbonding supplemented with mild reinforcement.

Check that shear reinforcement is properly spaced in the beams and that epoxy-coated bars are identified.

Shipping weight for each beam should be shown. When the difference between cambers for the Department supplied design and that shown on the shop drawings is significant, which can affect beam seat elevations, the fabricators should inform the Bridge Contractor for corrective measures.

Check for plastic drains, drip notch (if required), chamfers and other miscellaneous details. While a certain amount of minimum end block reinforcing is required in accordance with the Department's Standards, unusual end block conditions, particularly in post-tensioned or prepost-tensioned beams, requires additional reinforcement to inhibit end cracking. Frequently, designers ignore end block details, and it is up to the fabricators to provide a reinforcing cage for heavily stressed end blocks to inhibit cracking and crack propagation.

Check for post-tensioning sequence.

Insert sizes and locations in the beams must be verified, i.e., inserts for attachment of diaphragms, utility supports, lighting fixtures and, occasionally, guide rail connections.

(j) Laminated Shim Neoprene Pads

Check for size, total thickness, layers of neoprene, number of shims, hardness of neoprene, and skew and clip, if any. Make sure quantities of pads include sample pads for testing by the Laboratory Testing Section of the Innovation and Support Services Division. Pads must be pre-molded.

(k) Permanent Metal Deck Forms

Check against applicable BC-732M that the furnished formwork meets both, section modulus and moment of inertia requirements for the span length shown on the designs drawings (C. to C. stringers spacing less flange width, etc.). If a different pitch, depth or gauge thickness is used, the fabricators shall supply Manufacturer's computations for section modulus, area and moment of inertia for the forms.

Non-composite compression flanges must have flanges encased.

Where the bearing of the form is either at or below the bottom of the top flange, lateral support for the beams flanges may be assumed, and additional flange encasement is not required.

In continuous structures, welding of form supports to flanges in tension zones is not permitted. Design plans for rehabilitation structures frequently do not show tension zones and must be estimated or the data obtained from the designer. Check Special Provisions and design plans for restrictions on the use of metal deck forms. Generally, permanent metal deck forms are only permitted underneath structures. They are not intended for forming the outside of barriers, etc..

Review closure details at joints to assure compliance with permissible details. Details that could result in voids in the slab or difficult concrete placement should not be permitted.

Verify adjustments for camber and beams haunch, minimum gage requirements and configuration of forms.

(l) Metal Plate Culverts

Ascertain that the manufacturer furnishes the culvert specified on the design drawings and the Special Provision. Span and rise should be verified, as well as the gage of the material.

(m) Precast Concrete Culverts

The design drawings and Special Provisions will specify which design standard, if any, is applicable. Wall and slab thicknesses, as well as reinforcement, must be specified from the applicable standard. If the fabricator furnishes the design, a review of the design and approval by the District or the Bridge Design and Technology Division must precede the shop drawing review. Assure that proper 28-day strength, stripping and shipping strength of concrete are required prior to shipment from a fabrication plant.

(n) Timber Bridges

Check timber sizes, type and principal dimensions. Ascertain that proper hardware sizes are used and treatment of the timber is as specified in the Special Provisions.

(o) Impact Attenuators, R. C. Cribbing

These are proprietary items and shop drawings should be checked against the Design Drawings and Special Provisions to ensure that the item specified is in fact being furnished. Usually this will involve no more than a check of general configuration, i.e., layout, geometry and a check against catalogs furnished by the manufacturer.

(p) Proprietary Retaining Walls (Reinforced Earth, Retained Earth, Doublewall, etc.) and Anchored Pile Walls

Because of the Department's alternate bidding system, the design drawings may present these walls in concept only, with design parameters being given in the Special Provisions. In such cases, the shop drawings may also contain design aspects which must be reviewed. The reviewer should coordinate the shop drawing review with the District Bridge Engineer. Certain aspects may be subjected to a design review before the shop drawing review is started. Ensure that specified corrosion protective system is provided. The 28-day compressive strength is applicable generally for all precast elements.

(q) Quality Assurance

Adherence to the outlined procedures and guidelines will assure that structural materials specified by bridge designers in the contract documents are shown on fabrication drawings and reflect the intent of the design. It is up to the Department's inspection agents in the fabricating plants to assure that the materials and details shown on the shop drawings are, in fact, used during fabrication.

(r) Fabrication Errors

Frequently, errors are made during fabrication, especially in welded structures, which require changes from the original design. These changes must be approved either by the District or the Bridge Design and Technology Division, and be thoroughly documented. This documentation may require changing the design plans.

(s) Welding Procedure

Routine welding procedures are approved by the Structural Materials Engineer, Structural Materials Section, Bridge Design and Technology Division, Bureau of Project Delivery. Welding procedures must be reviewed as part of the shop drawing review, with particular attention to fracture-critical members.

(t) Erection Plans

Erection plans are usually part of the regular shop drawing review of structural steel or prestressed concrete shop drawings. Make sure that the erection plans provide data for setting expansion dam openings for temperature, and if found significant, end rotation.

(u) Alternate Design and Value Engineering

Prior to shop drawing review, plans and specifications for alternate designs and value engineered design allowed by the proposal must first be approved by the District or the Bridge Design and Technology Division. It is not the responsibility of the shop drawing reviewer to make judgment on equivalency of design, except for simple obviously equivalent substitutions of materials, welds, etc.

When proprietary items are used, it is important to ascertain that these items, whether they are taken from catalogs or Manufacturer's standard plans, have prior Department approval. In addition, the shop drawings shall be of standard size and the drafting requirement shall meet the drafting standards in Design Manual, Part 3.

(v) Epoxy-coated Bars

The requirement of additional development length for the epoxy-coated bars should be verified during shop drawing review.

(w) Some suppliers may be trying to utilize unapproved Strip Seal Expansion Dam Retainers, use of which will not assure long-term performance of Strip Seal Dams. Unapproved material should not be permitted through the shop drawing approval process.

Any vendor desiring to have his retainer approved must submit all details to the Bridge Design and Technology Division, Bureau of Project Delivery, for review and approval. Once it is approved and incorporated in Bulletin 15, it may be permitted in any contract. Any deviation requires a written approval from the Chief Bridge Engineer.

(x) In related matters on shop drawing reviews, the reviewer must ensure that all material, technical requirements and details indicated in the contract drawings, standards, construction specifications and design specifications (if applicable) are strictly adhered to. Any deviation must be approved in writing by the Director, Bureau of Project Delivery.

Sample
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION



(Use for (A) or (AAN) shop drawings after first review)

IN REPLY REFER TO

County _____

Date _____

Route _____

ECMS No. _____

Station _____, Segment / Offset _____ S- _____

Fabricator's Address:

Gentlemen:

With your letter of _____, you submitted shop drawings showing the
_____ for the subject structure.

Attached is one set of prints stamped (Accepted or Accepted as Noted).

Please return eight sets of prints of corrected drawings for our file and distribution.

Very truly yours,

Attachment

cc: Contractor: (One set if requested)

Sample
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION



(Use for (RFC) shop drawings)

IN REPLY REFER TO

County _____

Date _____

Route _____

ECMS No. _____

Station _____, Segment / Offset _____ S- _____

Fabricator's Address:

Gentlemen:

With your letter of _____, you submitted shop drawings showing the
_____ for the subject structure.

Attached is one set of prints stamped Returned for Corrections.

Please revise the shop drawings as required and return the corrected shop drawings for further review and acceptance.

Very truly yours,

Attachment

cc: Contractor: (One set if requested)

Sample
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION



(Use for distribution of metal deck forms.)

IN REPLY REFER TO

County _____

Date _____

Route _____

ECMS No. _____

Station _____, Segment / Offset _____ S- _____

Fabricator's Address:

Gentlemen:

With your letter of _____, you submitted shop drawings showing the
_____ for the subject structure. The drawings were (Accepted or Accepted as
Noted) on _____. We have attached _____ sets of prints.

Since there is no shop inspection of this material, we request that a thorough visual inspection be made at the job site.

Very truly yours,

Attachment

cc: General Contractor:

_____ (One Set)

District Bridge Engineer (One Set)

Sample
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION



(Use for distribution of all shop drawings, except metal deck forms.)

IN REPLY REFER TO

County _____

Date _____

Route _____

ECMS No. _____

Station _____, Segment / Offset _____ S- _____

Fabricator's Address:

Gentlemen:

With your letter of _____, you submitted shop drawings
 sheets _____ showing _____ for the subject structure.

The drawings were (Accepted or Accepted as Noted) on _____. We have attached
 _____ sets of prints. One set must be forwarded to the Department of Transportation Shop
 Inspector.

Very truly yours,

Attachments

cc: Department Shop Inspector

Contractor: (One Set)

Structural Materials Section (One Set)

Attention: _____, P.E.

District Bridge Engineer (One Set)



Sample
**PERFORMANCE REPORT
CONSULTING ENGINEERS**
Confidential

Date of Visit: _____

Visit Made By: _____

Consultant Represented By: _____

Consultant _____

Address _____

CONTRACTS Include Agreements being Completed and Current One when Completed													
LEGEND P - Poor F - Fair G - Good E - Excellent Answer all questions and check applicable block.	County												
	S.R. Rte. & Sec.												
	Type (Work)	___ Shop Drawings			___ Shop Drawings			___ Shop Drawings					
	Agreement No.												
	Status Work Completed	___ %			___ %			___ %					
	Est. Volume of Work	___ Sheets			___ Sheets			___ Sheets					
ATTITUDE AND COOPERATION		P	F	G	E	P	F	G	E	P	F	G	E
1. Cooperation with Department													
2. Cooperation with Other Agencies													
3. Attitude Toward Public													
4. Attitude Toward the Work													
5. Capability to Follow and Apply Specifications and Standards													
ORGANIZATION AND MANAGEMENT		P	F	G	E	P	F	G	E	P	F	G	E
1. Adequacy of Personnel													
2. Caliber of Personnel													
3. Caliber of Supervision													
4. Adequacy of Supervision and Management													
WORK PERFORMANCE		Under remarks, explain "No" answers with fault and reasons.											
		YES		NO		YES		NO		YES		NO	
1. Did Consultant start on time?													
2. Did Consultant prosecute work diligently?													
3. Did Consultant complete work on schedule?													
4. Were Consultant submissions complete?													
5. Was Consultant in possession of pertinent contract documents?													
6. Indicate status of visits, such as 1st, 2nd, 3rd for each project.													

Remarks:

File original with project file. cc:

Form completed by:

Date:

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX C – PAST AND CURRENT BRIDGE STANDARDS

The available standard drawings are divided into four major categories.

Utmost caution is advised in the use of Standard Drawings and Design Data discussed under a, b and c, since most of the data is based on the concept of working stress or load factor design methods.

(a) Standards for Old Bridges

These standards were assembled primarily for general inspection activities and for the analysis and rating of existing structures. They were issued as an internal publication (not listed in PennDOT Publication Sales Catalog) in five volumes by time period as follows:

- (1) Volume 1, from 1918 to 1930, published March 1983
- (2) Volume 2, from 1931 to 1940, published May 1983
- (3) Volume 3, from 1941 to 1960, published September 1983
- (4) Volume 4, from 1961 to 1965, published September 1989
- (5) Volume 5, from 1965 to 1972, published November 1989

These standards may be seen as a collection of subject matter for this time period containing construction details, as well as design data.

A limited number of these volumes are inventoried by the Bureau of Project Delivery, Bridge Design and Technology Division.

(b) Available Standard Drawings

Tables C-1 through C-8 present a comprehensive list of available standard drawings. It shall be noted, however, that many of the original designations have been changed. These tables can, therefore, be regarded as showing the historical changes of numbering and which standard drawings are most current.

In Table C-1, the drawing numbers in the columns on the left have an S-designation, and the equivalent ST-designation is shown in the columns on the right. The ST-symbol was used in the 1960's to designate standard drawing series.

Table C-2 lists the ST-100 series standards showing changes in approval dates, changes of drawing numbers, and elimination of individual drawings from the ST-series.

At a later date, the ST-100 series was divided into the ST-100 series for reinforced concrete and steel structures design-related items and into the ST-300 to BC-300 series for construction-related items (Tables C-3 and C-4).

In 1970, the above designations were changed from ST-100 to BD-100 series for design standards, and from ST-300 to BC-300 series for construction standards.

In January 1989, the BD-100 series and the BC-300 series were replaced by the BD-600 series and the BC-700 series, respectively.

The drawings listed in the attached tables were previously available under

- (1) Publication No. 3, BD-100 Bridge Design Standards
- (2) Publication No. 5, BC-300 Bridge Construction Standards
- (3) Publication No. 218, BD-600 Series Standards for Bridge Design (these supersede the BD-100 Series)
- (4) Publication No. 219, BC-700 Series Standards for Bridge Construction (these supersede the BC-300 Series)

Half-size drawings of the above are no longer available from the Department's Publication Sales Office. At present time, specific old standards can be requested from the Bridge Design and Technology Division (BDTD). Eventually all old standards will be added to the Bridge Standards Archives included on BDTD's Bridge Standards Web Site.

(c) Low Cost Bridge Standards - BLC Series

These standards are available as full-size, ready-to-use contract drawings (blank spaces provided for fill-in field data), as well as half-sized booklets containing a sample problem and instructions for the completion of the full-sized design drawings.

These standards have been developed for use on single span bridges to eliminate cumbersome calculations and drafting. To further increase productivity, the entire BLC package, as listed below, has been incorporated into the CADD system. A procedure for statewide usage has been developed and is available on the District level. The application of the BLC Series will do much to expedite the design and detailing of structures.

These plans are approved for use on projects involving liquid fuels funds and conventional funding proportions.

Small modifications may be needed when these plans are used for bridges on higher classes of roadway (additional details for guardrail transition, accommodations for approach slabs).

Short and medium single span structures on tangent alignment are particularly suited for these plans, whereas structures on curvature alignments may require the conventional custom design approach.

In the past, full-sized ready-to-use contract drawings (mylars) had been available through the District offices. Only half-sized booklets for Pub. 6M are available from the Department's Publication Sales Office.

The BLC Series applies to the following span ranges and types of structures:

<u>Pub. No.</u>	<u>Series</u>	<u>Span Range</u>	<u>Type of Structure</u>
17	BLC-500	(90'-130')	Steel Girders, P/S I-Beams, P/S Box Beams
49	BLC-510	(30'-90')	Steel Girders, P/S Box Beams
76	BLC-520	(18'-35')	Steel Beams, P/S Box Beams, P/C Channel Beams
114	BLC-530		Metal Plate Pipe Metal Plate Box/Arch R/C Rigid Frame, P/C R/C Box, R/C Box
130	BLC-540	(30'-90')	Timber on Steel Girders
134	BLC-550	18'-35'	Timber Bridges
6M	BLC-560M	18'-98'-5"	Hardwood Glulam Timber Bridge

(d) Current Standard Drawings

In January 1996, the metric standard drawings, BD-600M Series and BC-700M Series (shown in Tables C-9 and C-10, respectively) became the current standard drawings replacing BD-600 Series and BC-700 Series (shown in Tables C-7 and C-8). These drawings were produced with metric dimensions and are based on the LRFD Specification. However, the Metal Culverts and Concrete Pipe Standards, BD-635M and BD-636M, have been converted to metric dimensions, but their design is based on past AASHTO design specifications. The Overhead Sign Structures Standards (BD-641M through BD-645M) are based on the current AASHTO's Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals with the latest Interims. This design specification is based on the older working stress design criteria.

In December 1999 and June 2000, many of the metric standard drawings were revised to include U.S. Customary units in parenthesis after the metric values. These dual units standard drawings remain part of the BD-600M Series and BC-700M Series. The previous corresponding Standards (original U.S. Customary) were discontinued as indicated in Tables C-7 and C-8.

In December 2009, as a result of a decision by the AASHTO Subcommittee on Bridges and Structures to no longer publish SI unit specifications, the Department began the process of removing metric dimensions and notes from the BD-600 Series and BC-700M Series Standard Drawings.

The drawings listed in Table C-9 and C-10 are available in:

- (1) Publication No. 218M - BD-600M Series Standards for Bridge Design
- (2) Publication No. 219M - BC-700M Series Standards for Bridge Construction

Table C-1 – List of 1955-1968 Standards (OBSOLETE)

APPROVED DATE		NO.	DESCRIPTION	NO.	APPROVED DATE	
	3-1-61	S-2700 TO S-2703	R.C. SLAB BRIDGES	ST-100	2-25-65	
	11-30-62	SK-1188	R.C. T-BM. BRIDGES	ST-101	2-25-65	
	3-1-61	S-2711 TO S-2715	STEEL I-BM. BRIDGES			
	3-1-61	S-2716 TO S-2720	STEEL COMPOSITE I-BM. BRIDGES			
	8-2-63	S-2730 TO S-2736	STEEL I-BM. BRIDGES	ST-102	2-25-65	10-1-68
			STEEL I-BM. BRIDGES	ST-103	2-25-65	11-1-68
8-2-63	3-1-61	S-2724	STEEL I-BM. BRIDGES DIAPHRAGMS	ST-110	2-25-65	
8-2-63	3-1-61	S-2721	STEEL I-BM. BRIDGES BEARINGS	ST-111	2-25-65	1-2-68
8-2-63	3-1-61	S-2722	STEEL I-BM. BRIDGES PL. EXP. DAM	ST-112	2-25-65	
8-2-63	3-1-61 11-14-55	S-2722S-1593	STEEL I-BM. BRIDGES TOOTH EXP. DAM	ST-113	2-25-65	
8-2-63	3-1-61	S-2723	BRIDGE SCUPPERS			
			BRIDGE DRAINAGE	ST-114	2-25-65	
8-2-63	3-1-61	S-2727	R.C. ABUT. WITH BACKWALL	ST-120	2-25-65	
8-2-63	11-18-61	S-2728	R.C. ABUT. LAYOUT & DETAILS	ST-120	2-25-65	
8-2-63	3-1-61	S-2710	R.C. ABUT. WITHOUT BACKWALL	ST-121	2-25-65	
	7-5-61	S-2704	R.C. ABUT. FOR CONC. BRIDGES	ST-121	2-25-65	
	12-7-56	S-1612H	R.C. ABUT. DETAILS	ST-122	2-25-65	
8-2-63	3-1-61	S-2726	R.C. RETAINING WALLS	ST-123	2-25-65	
8-2-63	3-1-61	S-2725	ENDWALLS FOR M. PL. CULVERTS	ST-130	2-25-65	
	10-3-62	S-2729	R.C. BOX CULVERTS	ST-131	2-25-65	
S-1614C 10-17-62	S-1614B 5-23-58	S-1614 & S-1614A	PARAPET & PARAPET RAILING			
			ALUM. BRIDGE RAILING	ST-140	2-25-65	
	3-2-59	S-3361	STEEL BRIDGE RAILING	ST-141	2-25-65	
	7-10-63	S-6500	METAL CRIBBING, UNCOATED	ST-142	2-25-65	

Table C-1 – List of 1955-1968 Standards (OBSOLETE) (continued)

APPROVED DATE		NO.	DESCRIPTION	NO.	APPROVED DATE	
	7-10-63	S-6500A	METAL CRIBBING, COATED	ST-143	2-25-65	
			CONCRETE CRIBBING, I & II	ST-144	2-25-65	
	2-13-58	S-1613	ELECTRICAL DETAILS	ST-145	2-25-65	12-10-65
			ALUM. BRIDGE RAILING	ST-146	11-4-65	12-10-65
			STEEL BRIDGE RAILING	ST-147	11-4-65	12-10-65
3-1-66	3-5-65	S-5657*	PARAPET PROTECTIVE FENCE	ST-148	1-2-68	
			ELECTRICAL DETAILS	ST-149	21-2-68	
			ALUM. BRIDGE RAILING	ST-150	1-2-68	
			STEEL BRIDGE RAILING	ST-150	1-2-68	
	9-1-59	S-3361A	SIDEWALK RAILING			

H TITLE = WALL DETAILS

* TITLE = ALUM. CHAIN LINK FENCE

Table C-2 - ST-Series Standards (OBSOLETE)

DESCRIPTION	APPROVED DATE							
	MONTH	2	11	12	1	2	9	10
	DAY	25	4	10	2	5	23	1
	YEAR	65	65	65	68	68	68	68
TABLE OF CONTENTS	-							100
R.C. SLAB BRIDGES	100							E
DECK SLAB DETAILS	-							101
R.C. T-BM. BRIDGES	101							E
STEEL I-BM. BRIDGES	102							102
STEEL I-BM. BRIDGES	103							103
DIAPHRAGMS	110							111
BEARINGS	111				111			112
CLOSED JT. DETAILS	-							113
PL. EXP. DAM	112							114
TOOTH EXP. DAM	113							115
BRIDGE DRAINAGE	114							116
R.C. ABUT. WITH BACKWALL	120							121
R.C. ABUT. WITHOUT BACKWALL	121							122
R.C. ABUT. DETAILS	122							123
R.C. RETAINING WALLS	123							124
ENDWALLS FOR M. PL. CULVERTS	130							131
R.C. BOX CULVERTS	131							132
R.C. ARCH CULVERTS	-							133
ALUM. BRIDGE RAILING	140	146	146	150			312	
STEEL BRIDGE RAILING	141	147	147	151			313	
METAL CRIBBING	142					MC-2*		
METAL CRIBBING	143					MC-1*		
CONCRETE CRIBBING	144					CC-1*		
ELECTRICAL DETAILS	145		145	149			321	
PARAPET PROTECTIVE FENCE				148			301	

E = ELIMINATED

* = ELIMINATED FROM ST-SERIES AND INCLUDED IN THE ROADWAY STANDARD DRAWINGS

Table C-3 - ST-Series Standards (OBSOLETE)

DWG. NO.	DESCRIPTION	APPROVED DATE					
		MONTH	10	3	3	6	12
		DAY	1	18	24	12	17
		YEAR	68	69	69	69	69
ST-100	TABLE OF CONTENTS	X			X	X	
ST-101	DECK SLAB DETAILS	X				X	
ST-102	COMP. A36 STEEL I-BM. BRIDGES	X					
ST-103	COMP. A441 STEEL I-BM. BRIDGES	X					
ST-111	DIAPHRAGMS	X			X	X	
ST-112	BEARINGS	X				X	
ST-113	CLOSED JT. DETAILS	X					
ST-114	PLATE EXP. DAM	X				X	
ST-115	TOOTH EXP. DAM	X				X	
ST-116	BRIDGE DRAINAGE	X					
ST-121	R.C. ABUTMENT	X			X*		
ST-122	R.C. ABUTMENT	X			X*		
ST-123	R.C. ABUTMENT	X					
ST-124	R.C. RETAINING WALLS	X					
ST-131	METAL CULVERTS	X		X			
ST-132	R.C. BOX CULVERTS	X				X	
ST-133	R.C. ARCH CULVERTS	X				X	
ST-141	WATERSTOP DETAILS		X				

* SHEET 2 REVISED

Table C-4 - ST-Series Standards (OBSOLETE)

DWG. NO.	DESCRIPTION	APPROVED DATE			
		MONTH	9	6	12
		DAY	23	12	22
		YEAR	68	69	69
ST-300	TABLE OF CONTENTS	X	X		
ST-301	PROTECTIVE FENCE *	X	X		
ST-312	ALUM. BRIDGE RAILING	X	X		
ST-313	STEEL BRIDGE RAILING	X	X		
ST-314	ALUM. BRIDGE RAILING	X	X		
ST-315	STEEL BRIDGE RAILING	X	X		
ST-316	PEDESTRIAN RAILING				
ST-317	PEDESTRIAN-TRAFFIC BARRIER		X		
ST-321	ELECTRICAL DETAILS	X	X		
ST-322	LIGHTING POLE ANCHORAGE		X		
ST-331	PRECAST CEM. CONC. BLOCK SLOPE WALL	X			
ST-332	PERM. METAL DECK FORMS			X	

*TITLE CHANGED FROM PARAPET PROTECTIVE FENCE
TO PROTECTIVE FENCE, 6-12-69

Table C-5 (OBSOLETE)

DWG. NO.	DESCRIPTION		APPROVED DATE					
			9	9	11	3	5	9
		MONTH	1	1	1	1	12	28
		DAY	70	72	72	73	76	82
YEAR								
BD-100	INDEX		X				X	
BD-101	CONCRETE DECK SLAB		X					X
BD-102	COMP. A36 STEEL I-BM. BRIDGES		X					
BD-103	COMP. A441 STEEL I-BM. BRIDGE		X				X	
BD-104	MOMENT OF INERTIA GRAPHS		X				X	
BD-105	LIVE LOAD DEFLECTION NOMOGRAPH		X					
BD-105	ALLOWABLE FATIGUE STRESS IN STEEL & GIRDERS							
BD-111	CONCRETE DIAPHRAGM DETAILS		X				X	
BD-112	BEARING PEDESTAL DETAILS						X	
BD-121	R.C. ABUTMENTS WITH BACKWALL		X				X	
BD-122	R.C. ABUTMENTS WITH BACKWALL		X				X	
BD-123	R.C. ABUTMENTS WITHOUT BACKWALL		X				X	
BD-124	R.C. ABUTMENTS WITHOUT BACKWALL		X				X	
BD-125	R.C. ABUTMENTS MISCELLANEOUS DETAILS		X				X	
BD-126	R.C. RETAINING WALLS		X				X	
BD-131	METAL CULVERTS END WALL DETAILS		X				X	
BD-132	R.C. BOX CULVERTS		X					
BD-133	R.C. ARCH CULVERTS		X				X	
BD-201	PRESTRESSED CONCRETE BRIDGE STANDARDS					X		
BD-211	PRESTRESSED CONCRETE BRIDGE STANDARDS – ADJACENT BOX BEAMS			X				

Table C-6 – (OBSOLETE)

DWG. NO.	DESCRIPTION		APPROVED DATE																
			7	9	10	4	1	6	6	11	3	4	7	7	11	5	6	7	9
	MONTH		1	18	20	1	10	1	20	10	2	1	1	8	4	10	21	12	1
	DAY		70	70	70	71	72	72	80	80	81	81	81	81	81	82	82	82	83
BC-300	TABLE OF CONTENTS		X												X				
BC-301A	PROTECTIVE FENCE		X								X								
BC-309	PRECAST PARAPET												X						X
BC-311A	ALUMINUM PROTECTIVE BARRIER		X								X								
BC-312	ALUMINUM BRIDGE RAILING		X				X												
BC-313	STEEL BRIDGE RAILING		X				X												
BC-314	ALUMINUM BRIDGE RAILING		X					X											
BC-315	STEEL BRIDGE RAILING		X					X											
BC-316A	ALUMINUM PEDESTRIAN RAILING		X								X								
BC-317A	PEDESTRIAN-TRAFFIC BARRIER		X								X								
BC-318	GUARD RAIL CONNECTIONS TO PARAPETS		X					X											
BC-320	ALUM. OR STEEL BRIDGE HAND RAILING											X							
BC-321A	ELECTRICAL DETAILS		X						X										
BC-322A	LIGHTING POLE ANCHORAGE		X						X										
BC-331A	CEMENT CONC. BLOCK SLOPE WALL		X												X				
BC-332B	PERMANENT METAL DECK FORMS		X												X		X		
BC-334A	THREADED INSERT ANCHOR ASSEMBLY		X								X								
BC-335A	CONSTR. & EXPAN. JOINT DETAILS		X												X				

Table C-6 - (OBSOLETE) (continued)

DWG. NO.	DESCRIPTION		APPROVED DATE																
		MONTH	7	9	10	4	1	6	6	11	3	4	7	7	11	5	6	7	9
		DAY	1	18	20	1	10	1	20	10	2	1	1	8	4	10	21	12	1
		YEAR	70	70	70	71	72	72	80	80	81	81	81	81	81	82	82	82	83
BC-336A	REINF. BAR FABRICATION DETAILS	X												X					
BC-337A	RAILING ANCHOR SYSTEMS	X								X									
BC-338A	STRUC. MOUNTED GUARD RAIL & BARRIERS													X					
BC-339A	BRG. TERMINAL CONN. & INLET PLACEMENT													X					
BC-351	BRIDGE DRAINAGE	X																	
BC-352	CONCRETE DECK SLAB DETAILS	X			X														
BC-353	STEEL GIRDER DETAILS				X														
BC-354	STEEL DIAPHRAGMS	X																	
BC-355A	BEARINGS	X							X						X				
BC-356A	BEARINGS (FOR CURVED STL. BM. BRG.)								X										
BC-361	PLATE EXPANSION DAM	X																	
BC-362	TOOTH EXPANSION DAM	X																	
BC-363	ARMORED PREFORMED NEOPRENE COMPRESSION DAM			X															
BC-364	REINFORCED ELASTOMERIC EXPANSION DAM		X																
BC-365	CLOSED JOINT DETAILS				X														
BC-381	RANDOM STONE SLOPE WALL												X						
BC-391A	LOW COST BRIDGE																X		

Table C-7 – Listing of U.S. Customary BD Standard Drawings

Standard	Title	Signature Date
BD-600	INDEX OF STANDARDS FOR BRIDGE DESIGN	1/20/1989
		7/1/1993
		9/30/1994
		1/4/1999
		12/24/1999
		6/30/2000
		12/29/2000
		7/11/2001
		1/21/2003
BD-601	CONCRETE DECK SLAB, DESIGN & DETAILS FOR BEAM BRIDGES	1/20/1989
		8/10/1989
		7/1/1993
		9/30/1994
		6/9/1997
		12/24/1999
BD-602	SOUND BARRIER - CONCRETE POSTS ON SPREAD FOOTINGS, GEOMETRY AND REINFORCEMENT LAYOUT	3/8/1996
		1/2/1998
		6/30/2000
BD-604	GRID REINFORCED CONCRETE BRIDGE DECK	1/4/1999
		6/30/2000
BD-611	CONCRETE DIAPHRAGMS FOR STEEL I-BEAMS	7/1/1993
		9/30/1994
		12/24/1999
BD-621	R/C ABUTMENTS TYPICAL SECTIONS & DETAILS	4/4/1989
		7/1/1993
		8/1/1993
		9/30/1994
		12/24/1999
BD-622	R.C. ABUTMENTS WITH BACKWALLS, LAYOUT & DETAILS	4/4/1989
		7/1/1993
		9/30/1994
		6/30/2000
BD-624	R/C ABUTMENTS WITHOUT BACKWALLS, LAYOUT & DETAILS	4/4/1989
		7/1/1993
		9/30/1994
		6/30/2000
BD-625	R/C ABUTMENTS, MISC. DETAILS	4/4/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-631	END WALL DETAILS FOR METAL CULVERTS	1/19/1990
		7/1/1993
		9/30/1994
		6/30/2000
BD-632	R/C BOX CULVERTS	1/19/1990
		7/1/1993
		9/30/1994
		6/30/2000
BD-633	R/C ARCH CULVERTS	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-7 – Listing of U.S. Customary BD Standard Drawings (continued)

Standard	Title	Signature Date
BD-634	GABION END WALLS	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000
BD-635	DESIGN TABLES FOR METAL CULVERTS	1/31/1992
		7/1/1993
		9/30/1994
		6/30/2000
BD-636	REINFORCED CONCRETE PIPES	5/23/1994
		6/21/1994
		9/30/1994
		6/30/2000
BD-641	OVERHEAD SIGN STRUCTURES - CANTILEVER & CENTER-MOUNT STRUCTURES STRUT LENGTHS UP TO 40'	10/20/1995
		1/21/2003
BD-642	OVERHEAD SIGN STRUCTURES - TAPERED TUBE STRUCTURES SPANS FROM 30' TO 80'	8/1/1995
BD-643	OVERHEAD SIGN STRUCTURES - 2 POST PLANAR TRUSS SPANS FROM 30' TO 100'	8/1/1995
		7/11/2001
BD-644	OVERHEAD SIGN STRUCTURES - 2 POST AND 4 POST TRI-CHORD TRUSS SPANS FROM 60' TO 240'	8/1/1995
BD-651	P/S BRIDGES: TENDONS, DOWELS, SHEAR BLOCKS, DIAPHRAGMS & SKEW	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-652	P/S BRIDGES: BEAM SIZES & SECTION PROPERTIES	1/20/1989
		9/11/1989
		6/1/1991
		7/1/1993
		9/30/1994
		6/30/2000
BD-653	P/S BRIDGES: FRAMING & DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000
BD-654	P/S BRIDGES: ADJACENT BOX BEAM DETAILS	1/20/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		6/30/2000
BD-655	P/S BRIDGES, SIMPLE SPANS: TYPICAL SUPERSTR. SECTION	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-656	P/S BRIDGES, SIMPLE SPANS: TYPICAL LONGITUDINAL SECTIONS	1/20/1989
		9/11/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		6/30/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-7 – Listing of U.S. Customary BD Standard Drawings (continued)

Standard	Title	Signature Date
BD-657	P/S BRIDGES: ABUTMENTS (PLANS)	1/20/1989
		9/11/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-658	P/S BRIDGES: PIERS (PLANS)	1/20/1989
		9/11/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-659	P/S BRIDGES, SIMPLE SPANS: WATERPROOF. AT ABUTMENTS	1/20/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-660	P/S BRIDGES: DECK SLAB REINF.	1/20/1989
		9/11/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-661	P/S BOX BEAMS: REINF. - DETAILS	1/20/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-662	P/S I-BEAMS: REINF. - DETAILS	1/20/1989
		9/11/1989
		7/1/1993
		9/30/1994
		12/24/1999
BD-664	P/S I-BEAMS: CONTINUITY DETAILS FOR LIVE LOAD	1/20/1989
		9/11/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BD-665	P/S BOX BEAMS: CONTINUITY DETAILS FOR LIVE LOAD	1/20/1989
		9/11/1989
		1/16/1990
		6/1/1991
		7/1/1993
		9/30/1994
BD-666	P/S BEAMS: DEBONDING DATA & DETAILS	12/24/1999
		1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings

Standard	Title	Signature Date
BC-700	INDEX OF STANDARDS FOR BRIDGE CONSTRUCTION	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
		6/30/2000
		12/29/2000
BC-701	PROTECTIVE FENCE	11/15/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-702	SOUND BARRIER WALLS	9/3/1991
		7/1/1993
		9/30/1994
		1/2/1998
		6/30/2000
BC-711	ALUMINUM PROTECTIVE BARRIER	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-716	ALUMINUM PEDESTRIAN RAILING	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-720	ALUMINUM OR STEEL BRIDGE HAND RAILING	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-721	ELECTRICAL DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-722	LIGHTING POLE ANCHORAGE	1/20/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-731	CEMENT CONCRETE SLOPE WALL	1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-732	PERMANENT METAL DECK FORMS	1/20/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-734	ANCHOR SYSTEMS	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings (continued)

Standard	Title	Signature Date
BC-735	WALL CONSTRUCTION & EXPANSION JOINT DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-736	REINFORCEMENT BAR FABRICATION DETAILS	3/1/1989
		6/1/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-738	STRUCTURE MOUNTED GUIDE RAIL AND CONCRETE BARRIER	11/15/1989
		7/1/1993
		9/30/1994
		12/24/1999
BC-739	BRIDGE TERMINAL CONNECTION & INLET PLACEMENT	6/1/1991
		9/3/1991
		7/1/1993
		9/30/1994
		12/24/1999
BC-751	BRIDGE DRAINAGE DETAILS	1/20/1989
		11/15/1989
		6/1/1991
		7/1/1993
		9/30/1994
BC-752	CONCRETE DECK SLAB DETAILS	12/24/1999
		1/20/1989
		7/1/1993
		9/30/1994
BC-753	STEEL GIRDER DETAILS	12/24/1999
		1/20/1989
		6/1/1991
		7/1/1993
		9/30/1994
BC-754	STEEL DIAPHRAGMS	12/24/1999
		5/9/1989
		6/1/1991
		7/1/1993
		9/30/1994
BC-755	BEARINGS	12/24/1999
		1/20/1989
		7/1/1993
		9/30/1994
BC-757	PILE TIP REINFORCEMENT	6/30/2000
		1/20/1989
		6/1/1991
		12/2/1991
		7/1/1993
		9/30/1994
		12/29/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings (continued)

Standard	Title	Signature Date
BC-758	FIELD SPLICES FOR ROLLED BEAMS	1/20/1989
		5/9/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-762	TOOTH EXPANSION DAM	1/20/1989
		5/3/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-766	PREFORMED NEOPRENE COMPRESSION SEAL DAM	1/20/1989
		5/3/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-767	NEOPRENE STRIP SEAL DAM	1/20/1989
		5/3/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-781	RANDOM STONE SLOPE WALL	7/1/1993
		9/30/1994
		12/24/1999
BC-782	GABION SLOPE WALL	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000
BC-783	R.C. BRIDGE DECK REPAIRS	7/1/1993
		7/1/1994
		9/30/1994
		12/29/2000
BC-784	R.C. PARAPET MODIFICATIONS	7/1/1993
		9/30/1994
		12/29/2000
BC-785	BACKWALL REPLACEMENT	7/1/1993
		9/30/1994
		12/29/2000
BC-791	STANDARD LOW COST BRIDGE (P/C DECK & CULVERT)	1/20/1989
		7/1/1993
		7/1/1994
		9/30/1994
		12/29/2000
BC-792	CURB DRAIN DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		6/30/2000
BC-793	PRECAST CHANNEL BEAM	1/20/1989
		7/1/1993
		9/30/1994
		12/24/1999

Last Approval Date listed indicates when Standard was discontinued.

Table C-8 - Listing of U.S. Customary BC Standard Drawings (continued)

Standard	Title	Signature Date
BC-794	UTILITY ATTACHMENT & SUPPORT DETAILS	1/20/1989
		7/1/1993
		9/30/1994
		12/29/2000
BC-795	GEN. NOTES & LEGENDS FOR SOIL/ ROCK DESCRIPTION	1/20/1989
		1/20/1990
		7/1/1993
		9/30/1994
		6/30/2000
BC-798	PRECAST & R.C. BOX CULVERT	1/19/1990
		7/1/1993
		9/30/1994
		12/29/2000
BC-799	PREFABRICATED RETAINING WALL DETAILS	7/1/1993
		9/30/1994
		6/9/1997
		6/30/2000

Last Approval Date listed indicates when Standard was discontinued.

Table C-9 – Listing of Metric/Dual Unit BD Standard Drawings

Standard	Title	Signature Date	Status Indicator
BD-600M	INDEX OF STANDARDS FOR BRIDGE DESIGN	1/2/1996	
		1/4/1999	
		12/24/1999	
		6/30/2000	
		12/29/2000	
		7/11/2001	
		5/10/2002	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		12/15/2006	
		7/20/2007	
		12/29/2008	
		9/20/2010	
BD-601M	CONCRETE DECK SLAB	1/2/1996	
		12/24/1999	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		9/20/2010	
BD-602M	SOUND BARRIER WALLS Replaced by BD-676M to BD-679M	6/30/2000	D
		1/21/2003	
		7/29/2005	
BD-604M	GRID REINFORCED CONCRETE BRIDGE DECK	1/4/1999	
		6/30/2000	
		1/21/2003	
		9/20/2010	
BD-606M	PRECAST SLAB BRIDGES & CULVERTS	12/29/2000	D
		1/21/2003	
		7/24/2006	
		7/20/2007	
BD-609M	PA STRUCTURE MOUNTED GUIDE RAIL BARRIER	7/20/2007	
		12/29/2008	
		9/20/2010	
BD-610M	PA BRIDGE BARRIER	4/15/2004	
		7/24/2006	
		9/20/2010	
BD-611M	CONCRETE DIAPHRAGM DETAILS FOR STEEL I-BEAM STRUCTURES	12/24/1999	
		1/21/2003	
		7/29/2005	
		7/20/2007	
		9/20/2010	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Unit BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-612M	UTILITY ATTACHMENT TO SUPERSTRUCTURE	12/29/2000	
		1/21/2003	
		7/29/2005	
		9/20/2010	
BD-613M	HIGH LOAD MULTI-ROTATIONAL POT BEARINGS	5/10/2002	
		1/21/2003	
		7/20/2007	
		9/20/2010	
BD-614M	R.C. BARRIER MODIFICATION	12/24/1999	D
		12/29/2000	
		1/21/2003	
BD-615M	PA HT BRIDGE BARRIER	1/21/2003	
		7/29/2005	
		9/20/2010	
BD-616M	FIELD SPLICE	7/11/2001	
		1/21/2003	
		9/20/2010	
BD-617M	PA TYPE 10M BRIDGE BARRIER	1/21/2003	
		7/24/2006	
		7/20/2007	
		9/20/2010	
BD-618M	CONCRETE VERTICAL WALL BRIDGE BARRIER	1/21/2003	
		7/24/2006	
		7/20/2007	
		9/20/2010	
BD-620M	STEEL GIRDER BRIDGES LATERAL BRACING CRITERIA AND DETAILS	4/15/2004	
		9/20/2010	
BD-621M	REINFORCED CONCRETE ABUTMENTS	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/20/2007	
		9/20/2010	
BD-622M	R.C. ABUTMENTS WITH BACKWALL	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		9/20/2010	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Unit BD Standard Drawings (continued)

BD-624M	R.C. ABUTMENTS WITHOUT BACKWALL	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		9/20/2010	
BD-625M	WINGWALL LENGTH	1/2/1996	
		12/24/1999	
		1/21/2003	
		9/20/2010	
BD-627M	MOMENT SLABS	7/20/2007	
		9/20/2010	
BD-628M	BRIDGE APPROACH SLABS	12/15/2006	
		7/20/2007	
		9/20/2010	
BD-629M	REINFORCED CONCRETE PIERS (Originally issued as BD-659M)	7/24/2006	
		7/20/2007	
		9/20/2010	
BD-631M	END WALL DETAILS	1/2/1996	
		6/30/2000	
		12/29/2000	
		1/21/2003	
		7/24/2006	
		9/20/2010	
BD-632M	R.C. BOX CULVERT	1/2/1996	
		6/30/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		12/29/2008	
		9/20/2010	
BD-633M	R.C. ARCH DETAILS	1/2/1996	
		12/29/2000	
		1/21/2003	
		9/20/2010	
BD-634M	GABION END WALLS	1/2/1996	
		12/29/2000	
		1/21/2003	
		9/20/2010	
BD-635M	DESIGN TABLES FOR METAL CULVERTS	1/2/1996	
		6/30/2000	
		1/21/2003	
		9/20/2010	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Unit BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-636M	REINFORCED CONCRETE PIPES	1/2/1996	
		6/30/2000	
		1/21/2003	
		6/9/2010	
		9/20/2010	
BD-641M	OVERHEAD SIGN STRUCTURES - CANTILEVER & CENTER-MOUNT STRUCTURES STRUT LENGTHS UP TO 40'	1/21/2003	
		7/20/2007	
		9/20/2010	
BD-642M	OVERHEAD SIGN STRUCTURES - TAPERED TUBE STRUCTURES SPANS FROM 30' TO 80'	1/21/2003	
		7/20/2007	
		9/20/2010	
BD-643M	OVERHEAD SIGN STRUCTURES - 2 POST PLANAR TRUSS SPANS FROM 30' TO 100'	7/11/2001	
		1/21/2003	
		7/20/2007	
		9/20/2010	
BD-644M	OVERHEAD SIGN STRUCTURES- 2 POST & 4 POST TRI-CHORD TRUSS SPANS FROM 60' TO 240'	1/21/2003	
		7/20/2007	
		9/20/2010	
BD-645M	OVERHEAD SIGN STRUCTURES - 4 POST 4 CHORD TRUSS SPANS FROM 100' TO 200'	1/21/2003	
		12/15/2006	
		7/20/2007	
		9/20/2010	
BD-647M	MONOPIPE SIGN STRUCTURES	12/29/2008	
		9/20/2010	
BD-651M	REQUIREMENTS FOR TENDONS, DOWELS, SHEAR BLOCK, DIAPHRAGMS, SKEW LIMITATIONS AND BACKWALLS	1/2/1996	
		12/24/1999	
		12/29/2000	
		1/21/2003	
		7/29/2005	
		7/20/2007	
		9/20/2010	
BD-652M	PRESTRESSED BEAM SIZES AND SECTION PROPERTIES	1/2/1996	
		6/30/2000	
		1/21/2003	
		4/15/2004	
		7/20/2007	
		12/29/2008	
		9/20/2010	
BD-653M	TYPICAL FRAMING PLANS AND DETAILS	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		7/20/2007	
		9/20/2010	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Unit BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-654M	ADJACENT BOX BEAM DETAILS	1/2/1996	D
		6/30/2000	
BD-655M	TYPICAL SUPERSTRUCTURE SECTIONS	1/2/1996	
		12/24/1999	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		12/29/2008	
BD-656M	TYPICAL LONGITUDINAL SECTIONS	9/20/2010	
		1/2/1996	
		6/30/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
BD-657M	I-BEAM AND BOX BEAM BRIDGES	9/20/2010	
		12/24/1999	
		1/21/2003	
		7/24/2006	
		7/20/2007	
BD-658M	SHEAR BLOCK DETAILS AT PIER - P/S CONCRETE I-BEAMS AND BOX BEAM BRIDGES	9/20/2010	
		1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
BD-659M	TYPICAL ABUTMENT WATERPROOFING DETAILS Waterproofing Details moved to BC-788M REINFORCED CONCRETE PIERS (Revised to BD-629M)	7/24/2006	D
		1/2/1996	
		12/24/1999	
BD-660M	DECK SLAB AND STEEL REINFORCEMENT PLACEMENT	7/24/2006	
		1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		9/20/2010	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Unit BD Standard Drawings (continued)

BD-661M	BOX BEAM REINFORCEMENT DETAILS	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		9/20/2010	
BD-662M	I-BEAM AND PA BULB-TEE BEAM REINFORCEMENT DETAILS	1/2/1996	
		12/24/1999	
		12/29/2000	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		7/20/2007	
BD-664M	CONTINUITY FOR LIVE LOAD DETAILS - I-BEAM AND PA BULB-TEE BEAM BRIDGES	1/2/1996	
		12/24/1999	
		12/29/2000	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/20/2007	
		9/20/2010	
BD-665M	CONTINUITY FOR LIVE LOAD DETAILS - BOX BEAM BRIDGES	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
		7/20/2007	
		9/20/2010	
BD-667M	INTEGRAL ABUTMENT	12/24/1999	
		12/29/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
BD-668M	PRECAST CHANNEL BEAM BRIDGES	9/20/2010	
		12/24/1999	
		1/21/2003	
		7/20/2007	
BD-676M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE PANELS	9/20/2010	
		7/29/2005	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-9 – Listing of Metric/Dual Unit BD Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BD-677M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE POSTS	7/29/2005	
		9/20/2010	
BD-678M	GROUND MOUNTED SOUND BARRIERS - STEEL POSTS	7/29/2005	
		9/20/2010	
BD-679M	STRUCTURE MOUNTED SOUND BARRIER WALLS	7/29/2005	
		9/20/2010	
BD-680M	OFFSET SOUND BARRIER WALLS	7/29/2005	
		9/20/2010	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-10 – Listing of Metric/Dual Unit BC Standard Drawings

Standard	Title	Signature Date	Status Indicator
BC-700M	INDEX OF STANDARDS FOR BRIDGE CONSTRUCTION	1/2/1996	
		12/24/1999	
		6/30/2000	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		9/4/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		4/7/2008	
		12/29/2008	
		10/26/2010	
		3/21/2012	
BC-701M	PROTECTIVE FENCE	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		10/26/2010	
BC-702M	SOUND BARRIER WALLS Replaced by BC-776M to BC-780M	1/2/1996	D
		6/30/2000	
		1/21/2003	
		4/15/2004	
		7/29/2005	
BC-703M	THRIE-BEAM TO VERTICAL WALL BRIDGE BARRIER TRANSITION CONNECTION	1/21/2003	
		7/29/2005	
		10/26/2010	
		3/21/2012	
BC-707M	PA HT BRIDGE BARRIER	1/21/2003	
		7/29/2005	
		7/24/2006	
		10/26/2010	
		3/21/2012	
BC-708M	THRIE-BEAM TO PA TYPE 10M BRIDGE BARRIER TRANSITION CONNECTION	1/21/2003	
		7/29/2005	
		7/24/2006	
		10/26/2010	
		3/21/2012	
BC-709M	PA TYPE 10M BRIDGE BARRIER	1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		10/26/2010	
		3/21/2012	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-10 – Listing of Metric/Dual Unit BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-711M	ALUMINUM PROTECTIVE BARRIER	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/24/2006	
		10/26/2010	
BC-712M	THRIE BEAM TO PA BRIDGE BARRIER TRANSITION CONNECTION	4/15/2004	
		7/29/2005	
		10/26/2010	
BC-713M	PA BRIDGE BARRIER	4/15/2004	
		7/29/2005	
		7/24/2006	
		10/26/2010	
BC-716M	ALUMINUM PEDESTRIAN RAILING	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
		7/24/2006	
		10/26/2010	
		3/21/2012	
BC-718M	ALTERNATE RAILING DETAILS	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
		10/26/2010	
BC-719M	TEMPORARY CONCRETE BARRIER, STRUCTURE MOUNTED	1/21/2003	
		9/4/2003	
		4/15/2004	
		7/29/2005	
		12/29/2008	
		10/26/2010	
BC-720M	ALUMINUM OR STEEL BRIDGE HAND RAILING	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
		10/26/2010	
		3/21/2012	
BC-721M	ELECTRICAL DETAILS	1/2/1996	
		6/30/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		10/26/2010	

“D” indicates standard was discontinued on last Approval Date listed.

Table C-10 – Listing of Metric/Dual Unit BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-722M	LIGHTING POLE ANCHORAGE	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/24/2006	
		10/26/2010	
BC-723M	BRIDGE ANTI-ICING SYSTEM	7/29/2005	
		10/26/2010	
BC-731M	CEMENT CONCRETE SLOPE WALL	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
		10/26/2010	
		3/21/2012	
BC-732M	PERMANENT METAL DECK FORMS	1/2/1996	
		12/29/2000	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		10/26/2010	
BC-734M	ANCHOR SYSTEMS	1/2/1996	
		12/24/1999	
		1/21/2003	
		4/15/2004	
		10/26/2010	
BC-735M	WALL CONSTRUCTION & EXPANSION JOINT DETAILS	1/2/1996	
		12/29/2000	
		1/21/2003	
		10/26/2010	
BC-736M	REINFORCEMENT BAR FABRICATION DETAILS	1/2/1996	
		12/24/1999	
		1/21/2003	
		1/5/2007	
		10/26/2010	
		3/21/2012	
BC-738M	STRUCTURE MOUNTED GUIDE RAIL AND CONCRETE BARRIER	1/2/1996	D
		12/24/1999	

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Table C-10 – Listing of Metric/Dual Unit BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-739M	BRIDGE BARRIER TO GUIDE RAIL TRANSITION	1/2/1996	
		12/24/1999	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		12/29/2008	
		10/26/2010	
		3/21/2012	
BC-741M	OVERHEAD SIGN STRUCTURES - CANTILEVER & CENTER-MOUNT STRUCTURES STRUT LENGTHS UP TO 40'	1/21/2003	
		7/20/2007	
		10/26/2010	
BC-742M	OVERHEAD SIGN STRUCTURES - TAPERED TUBE STRUCTURES SPANS FROM 30' TO 80'	1/21/2003	
		7/20/2007	
		10/26/2010	
BC-743M	OVERHEAD SIGN STRUCTURES - 2 POST PLANAR TRUSS SPANS FROM 30' TO 100'	1/21/2003	
		7/20/2007	
		10/26/2010	
BC-744M	OVERHEAD SIGN STRUCTURES - 2 POST & 4 POST TRI-CHORD TRUSS SPANS FROM 60' TO 240'	1/21/2003	
		7/20/2007	
		10/26/2010	
BC-745M	OVERHEAD SIGN STRUCTURES - 4 POST 4 CHORD TRUSS SPANS FROM 100' TO 200'	1/21/2003	
		7/20/2007	
		10/26/2010	
BC-747M	MONOPIPE SIGN STRUCTURES	12/29/2008	
		10/26/2010	
BC-751M	BRIDGE DRAINAGE	1/2/1996	
		12/24/1999	
		7/11/2001	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		12/29/2008	
		10/26/2010	
BC-752M	CONCRETE DECK SLAB DETAILS	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
		10/26/2010	
		3/21/2012	

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Table C-10 – Listing of Metric/Dual Unit BC Standard Drawings (continued)

BC-753M	STEEL GIRDER DETAILS	1/2/1996	
		12/24/1999	
		12/29/2000	
		1/21/2003	
		7/29/2005	
		10/26/2010	
		3/21/2012	
BC-754M	STEEL DIAPHRAGMS	1/2/1996	
		12/24/1999	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		10/26/2010	
BC-755M	BEARINGS	1/2/1996	
		6/30/2000	
		1/21/2003	
		7/29/2005	
		12/29/2008	
		10/26/2010	
BC-757M	STEEL PILE TIP REINFORCEMENTS & SPLICES	1/2/1996	
		12/29/2000	
		1/21/2003	
		10/26/2010	
BC-762M	TOOTH EXPANSION DAM FOR PRESTRESSED CONCRETE & STEEL I-BEAM BRIDGES	1/2/1996	
		12/29/2000	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
BC-766M	PREFORMED NEOPRENE COMPRESSION SEAL JOINT FOR APPROACH SLABS	1/2/1996	
		12/29/2000	
		1/21/2003	
		7/20/2007	
		10/26/2010	
BC-767M	NEOPRENE STRIP SEAL DAM FOR PRESTRESSED CONCRETE & STEEL I-BEAM BRIDGES	6/30/2000	
		7/11/2001	
		1/21/2003	
		7/29/2005	
		7/24/2006	
		7/20/2007	
		10/26/2010	
BC-771M	STEEL INTERMEDIATE DIAPHRAGMS FOR PA. I-BEAM PRESTRESSED BRIDGES	1/2/1996	D
		12/29/2000	

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Table C-10 – Listing of Metric/Dual Unit BC Standard Drawings (continued)

BC-772M	PRESTRESSED CONCRETE BEAM BRACING	10/26/2010	
BC-775M	MISCELLANEOUS PRESTRESS DETAILS	1/2/1996	
		6/30/2000	
		7/11/2001	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/20/2007	
		10/26/2010	
		3/21/2012	
BC-776M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE PANELS	7/29/2005	
		10/26/2010	
BC-777M	GROUND MOUNTED SOUND BARRIERS - PRECAST CONCRETE POSTS	7/29/2005	
		10/26/2010	
BC-778M	GROUND MOUNTED SOUND BARRIERS - STEEL POSTS	7/29/2005	
		10/26/2010	
BC-779M	STRUCTURE MOUNTED SOUND BARRIER WALLS	7/29/2005	
		10/26/2010	
BC-780M	OFFSET SOUND BARRIER WALLS	7/29/2005	
		10/26/2010	
BC-781M	RANDOM STONE SLOPE WALL	1/2/1996	
		12/24/1999	
		7/11/2001	
		1/21/2003	
		10/26/2010	
		3/21/2012	
BC-782M	GABION SLOPE WALL DETAILS	1/2/1996	
		12/29/2000	
		1/21/2003	
		10/26/2010	
		3/21/2012	
BC-783M	REINFORCED CONCRETE REPAIR	1/2/1996	
		12/29/2000	
		1/21/2003	
		7/20/2007	
		10/26/2010	
		3/21/2012	
BC-784M	R.C. PARAPET MODIFICATIONS	1/2/1996	D
		12/29/2000	
BC-785M	BACKWALL MODIFICATION DETAILS	1/2/1996	D
		12/29/2000	

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Table C-10 – Listing of Metric/Dual Unit BC Standard Drawings (continued)

Standard	Title	Signature Date	Status Indicator
BC-788M	TYPICAL WATERPROOFING AND EXPANSION DETAILS	12/24/1999	
		6/30/2000	
		12/29/2000	
		1/21/2003	
		10/26/2010	
		3/21/2012	
BC-791M	LOW COST BRIDGE	1/2/1996	D
		12/29/2000	
BC-792M	CURB DRAIN DETAILS	1/2/1996	D
		6/30/2000	
BC-793M	PRECAST CHANNEL BEAM BRIDGES	1/2/1996	D
		12/24/1999	
BC-794M	UTILITY ATTACHMENT & SUPPORT DETAILS	1/2/1996	
		12/29/2000	
		1/21/2003	
		10/26/2010	
BC-795M	GENERAL NOTES AND LEGENDS FOR SOIL / ROCK DESCRIPTION	1/2/1996	
		6/30/2000	
		1/21/2003	
		10/26/2010	
BC-798M	MECHANICAL CONNECTION DETAILS	1/2/1996	
		12/29/2000	
		7/11/2001	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		10/26/2010	
		3/21/2012	
BC-799M	MECHANICALLY STABILIZED EARTH RETAINING WALLS	6/30/2000	
		12/29/2000	
		1/21/2003	
		4/15/2004	
		7/29/2005	
		7/24/2006	
		4/7/2008	
		10/26/2010	
		3/21/2012	

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

**APPENDIX D – CHECKLIST FOR CONTRACTOR'S DESIGN
ALTERNATE - CONCEPTUAL DESIGN REVIEW**

Contractor's Design Alternates -- Conceptual Design

The following items shall be addressed during conceptual design reviews:

- (a) The completeness of the submitted package.
- (b) The inclusion of comments from the reviewing Consultant and the District. Central Office review shall be complete by the time the comments from the District and Consultant come in if simultaneous submissions are made.
- (c) The computer program used in the analysis - state clearly which program is being used. The agreement of Design/Analysis Methodology with Special Provision Design Methodology (line girder, 2-D, 3-D).
- (d) The pier/abutment arrangement - has it been changed from the original scheme (check Special Provisions).
- (e) The provision of core boring data if required by the contract.
- (f) The provision of substructure design data - approved soil bearing resistance parameters, and approved pile resistance parameters shall be same as in original design.
- (g) The clear indication of pouring sequence - conceptual drawings shall show it.
- (h) Debonding - the use of up-to-date criteria.
- (i) Bearings - have they been altered. (If altered, is design and type of alternate bearing approved by the Department)
- (j) Joint layout - has it been altered. (Number of joints may be decreased, but not increased)
- (k) Diaphragm arrangement is to be checked closely; it shall meet LRFD requirements.
- (l) Approval letter must contain the following sentence: "Approval of concept is subject to review of detail plans/design."
- (m) Utility arrangement is also to be compared to original design.
- (n) Type of expansion joints shall be approved at conceptual stage.
- (o) Expansion joints - check versus standard drawings.
- (p) The submission shall be treated as a combination of TS&L and foundation with emphasized attentions toward the original design.
- (q) Deck and end structure drainage shall be the same as in as-designed bridge plans.
- (r) Approval letter shall contain all TS&L items, girder spacing, foundation items (not as detailed as for regular approval) and design methodology.
- (s) Construction methods for untypical structures.
- (t) Awareness of problem areas - ask for early partial submittals on problem areas.
- (u) Flood clearance.
- (v) Commitment of the Contractor to his contract pile quantity in the approval letter, if alternate design uses as-designed piles. Pile quantity shall be approved in accordance with the Special Provisions. Contractor shall be notified if the required quantities are different from his bid quantity so he can withdraw his concept if he must install a significant amount of piling without compensation.

- (w) Steel structures - verify that web buckling criteria and deck pouring sequence were examined as required.
- (x) Corrosion protection of substructure in accordance with current Department criteria.
- (y) Pile driving requirements shall be checked if as-designed piles were not used.
- (z) Introduction of fatigue prone details - do not approve without approval of Chief Bridge Engineer.
- (aa) Fracture-critical concepts - do not allow unless included in as-designed bridge plans.
- (bb) Department criteria shall be checked to determine whether a backwall is required or an end diaphragm may be provided.
- (cc) A list must accompany the submissions that states all variations from the as-designed bridge plans. Anything not listed must conform to the as-designed plans.
- (dd) If light poles are located on the structure, the rotation and deflection of the structure at the light pole locations for the Contractor's alternate must be less than the structure rotation and deflection at the light pole locations for the as-designed structure. Calculations must show this.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
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PART 4

VOLUME 2

APPENDIX E – BRIDGE MANAGEMENT WHITE PAPER

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1.0 ASSET DEFINITION

PennDOT owns approximately 25,250 bridges greater than or equal to 8 feet in length, which is equivalent to more than 110 million square feet (MSF) of deck area. Of the 25,250 bridges, approximately 15,445 (108 MSF) have an opening along the centerline of roadway greater than 20 feet, which is the Federal Highway Administration's (FHWA) definition of a bridge. Approximately 2,620 of the 25,250 bridges are on the Interstate system and 3,400 more are on the National Highway System. PennDOT's bridge inventory consists of almost all structure types including significant arch or truss bridges crossing major rivers, but the predominant bridge type is an I-shape girder constructed of either steel or concrete. As an indication of the value of the bridge assets, the replacement cost of all bridges is more than \$33 billion for the construction alone. Bridges comprise the majority of structural assets; however, other structural assets that are managed (inventoried and inspected) are sign structures, walls and tunnels.

2.0 GOALS AND POLICIES

The overarching goals of PennDOT's bridge program include:

- Ensuring the bridges are safe for the efficient movement of people and goods.
- Reducing the backlog of bridge deficiencies to the national average, as measured by the deck area of bridges that are Structurally Deficient.
- Using good practice in design, construction, and maintenance to sustain the continual improvement of our bridges in a cost effective manner and move towards a bridge life of 100 years.

Our holistic approach to bridge management includes timely inspection, load rating analysis, on-demand structural repairs, preservation, and routine maintenance to maximize the life of a bridge. For bridge improvements, PennDOT is incorporating a design approach and details that will result in a 100-year service life. (See Table 2.0-1)

The policies governing the bridge program are:

Safety - Inspection and load rating analysis

Federal Requirement - National Bridge Inspection Standards (NBIS), Code of Federal Regulations Chapter 23 Part 650, Subpart c.

PennDOT Requirements - Publication 238 Bridge Safety Inspection Manual

100 Year Life - Design and Construction

Federal Requirement - AASHTO LRFD Bridge Design Specification, Title 23 Section 144 Highway Bridge Program

PennDOT Requirements - Publication 15M Design Manual Part 4, Publication 218M Standard Drawings for Bridge Design (BD-600M series), Publication 219M Standard Drawings for Bridge Construction (BC-700M series) and Publication 408

Bridge Improvements and Preservation - Planning and Programming

Federal Requirement - Federal Highway Bridge Program (HBP), Code of Federal Regulations Chapter 23 Part 650, Subpart d.

Maintenance - Address critical structural repairs, repair of leaking deck joints; routine cleaning and flushing of joints, decks, scuppers, and beam seats.

Federal Requirement - 23 USC (United States Code) 116

PennDOT Requirements - Publication 55 Procedures and Standards for Bridge Maintenance, Publication 23 Maintenance Manual, ASHMA Budget Allocation

Table 2.0-1 - 100 Year Bridge Life Matrix

Forces	Design and Construction Solutions	Era of Implementation	Construction Cost	Construction Quality	Inspections	Maintenance/Preservation	Rehabilitation
Corrosion	1. Minimize number of Deck Joints 2. Eliminate deck joints. No joints on bridges (put joints beyond approach slabs for bridge skews greater than 70°)	Mid 80s • Implement policy late 2006	Minimal net increase	Construction Quality - Do not accept defective material such as buckled steel or cracked prestressed beams. Install properly designed riprap, install piles in accordance with specifications, set bottom of footing elevation on proper bearing material	Focus on beam ends and bearing areas Element level inspection and reporting (BMS II) Deck concrete sounding Perform accurate and timely inspections	Repair leaking joints Clean/flush deck, scupper, bearing areas and beam seats Deck Patching Latex overlays extends deck life 20 years Membrane waterproofing and bituminous overlays Alternate deck joint systems - low cost, easily maintained and repairable such as XJS for retrofits Spot and Zone Paint Perform identified maintenance/repairs of structural components Beam repairs	Replace ruptured joint glands Hydro-demolition and deep overlay Replace decks after 40-50 years Blast and Repaint Steel Bridges (Environmental challenge for lead base paint)
	Eliminate Barrier Deflection Joints	Mid 80s	Minimal net increase				
	Epoxy coated deck reinforcement	Early to mid 70s	\$2000-\$5000 per bridge (\$0.05-\$0.10/lbs.)				
	Prestressed Beams with epoxy coated reinforcement	Mid 80s	Minimal increase				
	Prestressed Beams exposed to salt spray from traffic underneath chemical additives and sealants	• Under development	• Under development				
	Steel Beams: A588 Weathering Steel in certain environmental conditions with painted ends under joints Bridge Painting - Inorganic zinc rich paint system	Mid to late 80s Mid to late 80s	\$0.05/lbs \$0.15/lbs				
	Epoxy coated J-bars (Abutments and Piers)	Mid 80s	\$500-\$1000 per bridge (\$0.05-\$0.10/lbs.)				
Fatigue	Improved Structural Detailing - Welding of connection plates	Mid 80s	\$1500-\$2000 per bridge		Fracture critical bridge inspection is critical Major structures susceptible connections at ends of floorbeams Pre mid 80s constructed bridges - web distortion	Drill tips of fatigue cracks prior to significant crack growth	Address floorbeam connections for major structures Address out of plane distortion connection
	Design Equations for 100 year life	Mid to late 80s	Minimal increase in material cost				
Scour	Properly designed riprap	Early 90s	\$15,000 per bridge		Footing undermining of older bridges Movement of riprap Stream bed scour holes	Install counter scour measures - riprap, stream bed paving, curtain walls Removal of debris in streams	Install counter scour measures – riprap Removal of debris in streams
	Appropriate Bottom footing elevations	Early 90s	\$30,000 per substructure				

Ongoing Activities

Eliminate use of Adjacent Noncomposite Prestressed Box Beams bridge

Maintenance – Develop new performance measurement metrics.

All structures must have redundancy (implemented in 1980's)

3.0 INVENTORY

Inventory and location data is collected by PennDOT and stored in its Bridge Management System 2 (BMS2) for all state-owned bridges greater than or equal to 8 feet in length. PennDOT collects and stores all data as required by the FHWA's "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," for compliance with the Federal NBIS regulations. Examples of the inventory data are: Structure Type, Structure Length, Structure Width, Deck Type, Year Built, Latitude, Longitude, Features Intersected Data, Hydrologic Data and Posting Data. All bridge inventory data is entered into BMS2 prior to the initial inspection. All bridges are inspected at a maximum frequency of every 2 years, and changes in the inventory data are noted and collected in the field.

The Department also collects additional information for bridges that is not required by FHWA. Examples include: Design and Shop Drawing Numbers, State Senatorial and Congressional Districts, Detailed Railroad Information, Design and Material Information and Fracture Critical Information. Information is also collected for sign structures, walls and tunnels and is stored in BMS2.

PennDOT implemented its new Pontis based BMS2 in November 2006. New data fields have been added to store more information in order to meet the Department's needs. The new fields include: additional Inspection Planning Information, Material Testing Results, and additional Load Rating Information.

4.0 CONDITION ASSESSMENT AND PERFORMANCE MODELING

Inspection and appraisal data for bridges is also collected by PennDOT and stored in BMS2 for all state-owned bridges greater than or equal to 8 feet in length. PennDOT collects and stores all inspection data as required by the FHWA's "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges".

Examples of the inspection data quantifying the condition of a bridge are:

- Superstructure Rating
- Substructure Rating
- Deck Rating
- Observed Scour Rating
- Load Capacity Rating

In addition to the FHWA required inspection data, PennDOT collects more detailed condition data on its bridges, such as Paint Condition.

The Department also requires bridge inspectors to recommend maintenance items from a pre-determined list of repair items.

The frequency of bridge inspections is every 2 years or less, depending on the condition of the structure. Inspectors collect and enter the inspection data into electronic data collectors (EDC). The use of EDCs has greatly expedited the transfer of data from the field to the office through an automated process. Once the data is entered and then approved by the Bridge Inspection Supervisors, it is available in BMS2 for Department personnel to run reports for their use. District Bridge Units typically query selected inspection, inventory and maintenance data items for their planning and programming of bridge work.

The Department has initiated a process to develop predictive models using Pontis/BMS2. The first step in this process was to define bridge elements at a more detailed level than those currently inspected, and train PennDOT inspectors to conduct element-level inspections. The Department conducted its initial element-level inspections in the spring of 2007. Accumulating condition history data on homogeneous elements of structures will allow the Department to build predictive deterioration models. The next steps are to develop interim models based on expert judgment that can be improved over time. These predictive models are required in order to utilize the simulation and optimization features of Pontis.

5.0 ALTERNATIVES EVALUATION AND OPTIMIZATION (DECISION MAKING PROCESSES)

Bridge improvements are identified and prioritized at the District level based on structural condition, sufficiency rating and repair needs identified from recent inspections.

Bridge preservation projects are prioritized based on work type (scour correction, leaking joints, elimination of the most urgent maintenance priorities – e.g. maintenance items with priorities of 0's and 1's) and road category as defined by the Business Plan Network.

For bridge improvements the funding is developed with Planning Organizations, Transportation Improvement Program (TIP), Bridge Bill and Twelve Year Plan (TYP).

The Pontis capabilities of BMS2 will be used in the future for alternatives evaluation and optimization.

6.0 IMPLEMENTATION (WHAT IS DONE AND HOW)

The major treatment categories for bridges are as follows:

- **Maintenance** - on-demand repairs to restore damaged or deteriorated members to a minimum level of safety performance, or small-scale preventive maintenance actions to ensure the bridge members realize their expected service life. Maintenance is generally limited to cleaning activities and smaller scale preventative maintenance items (e.g. sealing joints, drainage repairs) at each bridge. Work may be done by Department forces or by outside contractors.
- **Preservation** - actions to extend the life of a bridge without improving bridge functionality or performance (keeping good bridges good). The scale of work tends to be much larger than Maintenance work, with a longer extension of bridge life expected. Examples of preservation work items include deck joint replacement, beam end repairs, deck overlays, painting and fatigue retrofits. Most preservation projects are performed by contractors; some are performed by Department Maintenance crews.
- **Improvement** - major rehabilitation or replacement of bridges that require an improvement in bridge safety and/or performance over an extended period of time (30+ years). The need for improvement may be dictated by structural concerns and/or functional needs of the highway system. Improvements include structure replacement or major rehabilitation. Most Improvement projects are typically performed by contractors, but there is an occasional bridge replacement or rehabilitation performed by County Maintenance crews.

All bridge work to be performed takes into account the needs of other assets (e.g. roadways, signs, etc.) so that work for these items may be coordinated and implemented at the same time to reduce costs (e.g. mobilization, traffic control, etc.).

7.0 PERFORMANCE MONITORING

The following are measures and targets:

- Achieve the Bridge Program Lets in Dollar Amount - Set Yearly by Executive Staff
- Achieve the Number of Bridge Program Lets - Set Yearly by Executive Staff
- Percent of Structurally Deficient (SD) Bridges, Measured by Deck Area for State-Owned Bridges ≥ 8 feet - Reduce SD Deck Area to national average
- Completion of 0 and 1 Maintenance Priorities for Bridge Maintenance Category – Goal is to complete priority maintenance items in accordance with timeframes in Publication 238
- Compliance with NBIS Inspection Frequency - 100% within 24 months, no inspection is beyond 25 months
- Monitor Bridge Analysis Backlog - 60 day cycle time on load rating analysis

Improvements to performance monitoring under consideration are: 85% of bridge improvement construction let dollars are SD bridges, improvement of Interstate SD bridges, project delivery streamlining, effectiveness of bridge preservation, quality of bridge inspections and quality of load rating analysis.

8.0 NEXT STEPS

Next steps for improved bridge management include:

- **Inspection - Element Level Data Collection and Support for Predictive Modeling**
Bridge inspections will collect more refined and quantified condition information such as lineal feet of deteriorated beams or square feet of deteriorated deck. This refined data will be used in predictive modeling. PennDOT has begun the task of collecting element level condition state assessments.
The model will predict condition performance factors and costs for an optimal mix of various actions (including preservation, rehabilitation and improvement) over time. A minimum level of maintenance is assumed. Some maintenance (e.g. cleaning, etc.) and preservation treatments (e.g. deck seals) can be achieved through specified treatment cycles. During the initial phase of element data collection, the Department will have sufficient time to develop its agency's rules for modeling. Default Pontis condition models need to be modified to better represent PA bridges. Initially, the PA models may be developed based on expert judgment that can be improved over time from actual element condition data. These predictive models are required in order to utilize the simulation and optimization features of Pontis.

- **Risk Assessment**

The Department is evaluating the risk to bridge safety and long-term performance by several factors, including scour, fracture-critical members, load capacity, maintenance needs, as well as component condition. This assessment is being done to ensure that work critical to a bridge's performance is adequately considered in new bridge investment optimization methods and programming decisions.

- **Interstate Management Program**

The Interstate Management Program Guidelines have been published which include baseline indicators and target values. The Districts will use this information in project planning and programming, and we will monitor the program and resultant system improvement.

- **Support for Enterprise Asset Management**

Bridge analyses is to provide needed bridge costs and performance information, including "what-if" scenarios, to assist in trade-off analysis between asset categories. Analysis methodology must be consistent with Department standards.

9.0 IMPLEMENTATION AND CONTINUATION OF NEXT STEPS

To ensure the successful implementation and continuation of the initiatives for asset management of bridges, the following steps need to be taken and/or continued:

9.1 Communication

The intent and plan for Asset Management of Bridges will be communicated throughout the Department in the following forums:

- Annual Bridge Program Audits of the Districts
Conducted by the Chief Bridge Engineer to review accomplishments, programmed projects, bridge needs and other operational issues with the District Bridge Engineer.
- Various Department-wide leadership meetings held throughout the year
Including: Bridge Engineers meeting, Structure Control/Bridge Engineers meetings, Bridge Maintenance Coordinators meetings, selected DE/ADE meetings, bridge inspection training, manager meetings for other PennDOT systems (RMS, Plant Maintenance, etc.), Construction Management Training Workshops.

9.2 Implementation

The various components of Asset Management of Bridges will be incorporated into appropriate sections of the following Department documents:

- Design Manual Part 4 – Structures (Pub 15M)
- Bridge Safety Inspection Manual (Pub 238)
- Software – The Department is to continue to develop and improve state-of-art software for various activities, such as:
 - Design – BRADD, other bridge design software
 - Planning and Programming – BMS2 (Pontis)
 - Asset Management – Pending development of a system
 - Inspection – I Forms

9.3 Quality Assurance

Various aspects of the quality of the overall management of PA bridges program are reviewed for quality in the following ways:

- Design – QA review of District design processes/plans is performed by BDTD staff each year.
- Bridge Programming and Delivery – the projects selected and delivered by the Districts for the TIP are monitored on an on-going basis by the Asset Management Division to ensure program meets overall bridge initiatives and schedule.
- Inspection – Statewide Inspection QA program reviews selected bridge inventory, inspection, and capacity rating data from bridges in each District through blind inspections by consultant. Compliance with NBIS inspection frequency is checked monthly by the Asset Management Division.

9.4 Reporting and Monitoring Plans

Various aspects of the management of the bridges are monitored and reported through:

- Annual Business Plans
- Dashboard and Scorecard Performance measure

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX F – REFERENCE MATERIAL FOR LATERALLY LOADED PILES AND DRILLED SHAFTS

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1.1 LATERAL LOADS ON PILES

The following Appendix lists information on the development of design curves used to determine the lateral deflection and moment on a pile when given a soil profile and a lateral load. The curves were originally developed for service load design. Since the method used was not dependant on the design method used (LRFD or Service Load Design), the curves can be used with any design method as long as the specified units are consistent with the loads used. The design curves should be used as described in D10.7.3.12.2P.

1.1.1 General

Lateral loads for piles supporting retaining walls, abutments, and piers are generally governed by deflections rather than an ultimate lateral load. Two approaches are available for analyzing laterally loaded piles [1,2,3]. In the Winkler, or subgrade reaction approach, the soil is modeled as a series of unconnected springs. In the elastic continuum approach the soil is assumed to be an ideal elastic material. Both methods of analysis deal with the lateral resistance of soil and the variation of resistance with depth as the pile is pushed against the soil. The major uncertainties of both methods are related to the assumptions used to determine the lateral soil resistance, which is represented by the definition of the soil spring in the Winkler approach and the modulus of elasticity for the elastic continuum approach. At this time the Winkler approach is considered more practical and versatile for routine analyses. The key factor in the successful application of the Winkler approach is the definition of the soil springs.

1.1.2 P-Y Curves and the COM624G Computer Program

The behavior of the soil spring at any point along the length of the pile can be modeled by the relationship between the soil pressure per unit length (p) and lateral deflection (y) at that point. This relationship is typically expressed as a p - y curve and is linear only for a small range of deflections. In general, p - y curves are nonlinear and depend on several parameters, including depth, shear strength, coefficient of friction, unit weight, and the number of load cycles.

A computer program has been developed by Reese and Sullivan to analyze the behavior of a laterally loaded pile utilizing p - y curves to model the soils springs [4,5]. A modified version of this program [6], the COM624G, developed to be used with an IBM Personal Computer, has been used to develop the design charts in this manual. The COM624G program includes all the features of the original Reese program. (Note that the current version of the program is COM624P. LPILE 5.0 is an alternate lateral analysis computer program approved for use.)

In the COM624G program the pile length is divided into many segments, each acting independently. P - Y curves are developed for each segment. The governing differential equation is then solved by the finite difference method. An iterative procedure is used until satisfactory compatibility is obtained between the predicted behavior of the soil and the load-deflection relationships required by an elastic pile.

The COM624G program is equipped to develop p - y curves for the following soil conditions:

- (a) Soft clay below the water table
- (b) Stiff clay below the water table
- (c) Stiff clay above the water table
- (d) Sand, above or below the water table

As an example, Figure 1.1.2-1 contains the plots of p - y curves computed for various depths for a loose to medium dense sand ($\phi=32$) above the water table under static and cyclic loading. For sands, the slope of the initial straight line portion of the curve is the secant modulus of soil reaction, E_s , where:

$$E_s = \frac{-P}{y}$$

and E_s is related to depth by the constant k :

$$E_s = kx$$

It can be seen from the p - y curve plots that the extent of the straight line portion of the curve varies with depth and mode of loading (cyclic or static). The level portion of the curves represents the yield point of the soil at a given depth. For this case, a cyclic loading condition only influences the p - y curves near the ground surface.

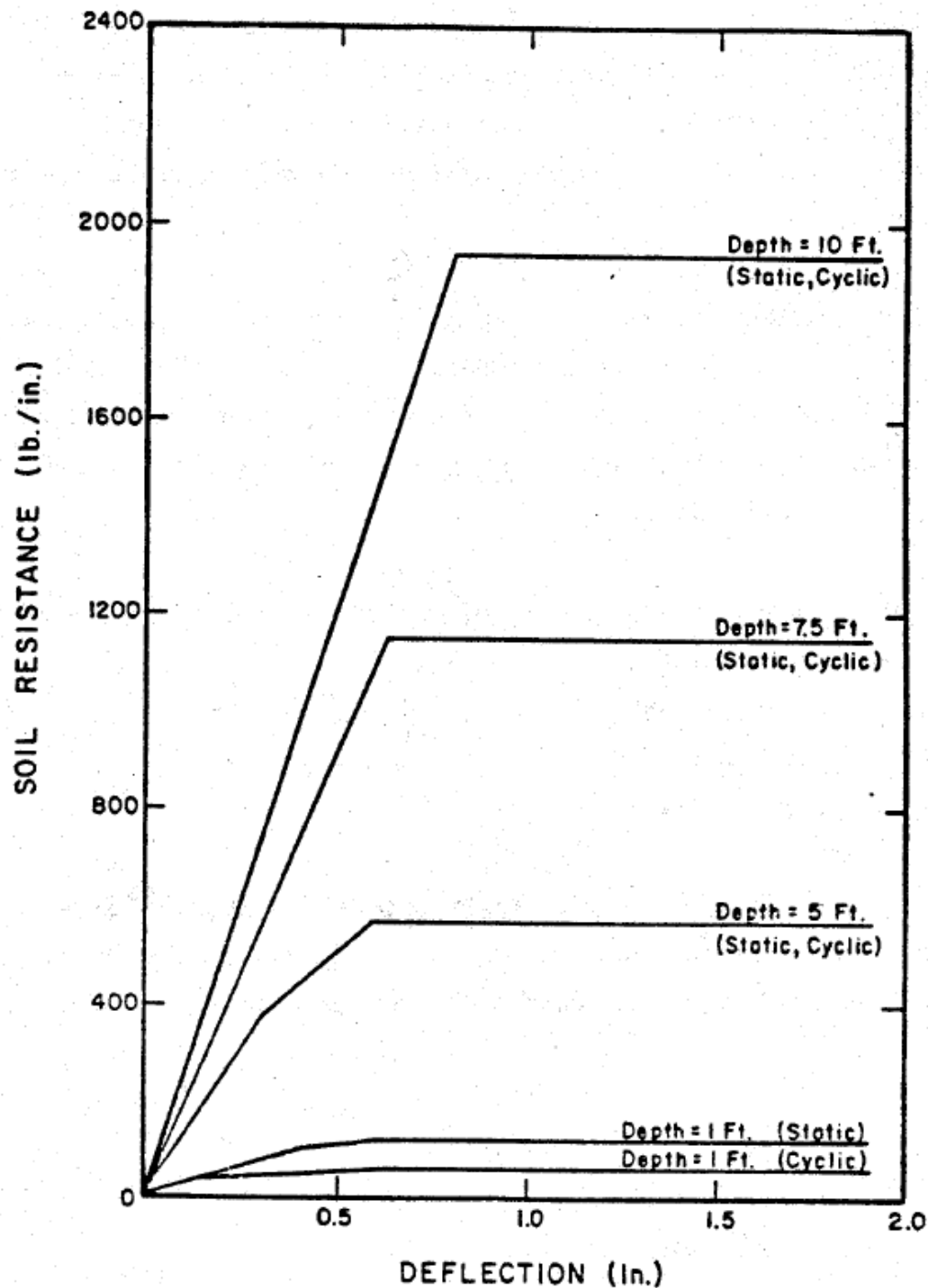


Figure 1.1.2-1 - P-Y Curves in Loose Sand

The methods used to develop the p-y curves in the program are empirical, based on the results of a limited number of load tests. Other p-y curves can be developed and input into the program. The COM624G program is also equipped to evaluate fixed, free, or partially restrained pile head; static or cyclic loading; layered soils; axial loads; and variations in the flexural rigidity (EI) of the pile with depth.

1.1.3 Parametric Studies

The effects of changing various parameters were studied prior to developing the design cases. The results of these studies may be useful in applying the computed COM624G results to actual conditions. A 16-inch pipe pile was generally analyzed for two soil conditions, loose sand ($\phi = 32$ degrees, $\gamma = 110$ pcf, and $k = 35$ kcf) and soft clay ($c = 500$ psf, $\gamma = 100$ pcf, $\epsilon_{50} = .02$, and $k = 52$ kcf). The constant, k , and the strain corresponding to one-half the maximum principal stress difference, ϵ_{50} , are as recommended in the program User's Manual [4]. Results of the parametric studies are as follows:

- (a) Length-Pile lengths were varied between 15 and 100 ft. for lateral loads of 20 and 25 kips, and pile head fixity of zero (free-head) and 100 percent (fixed-head). It was found that ground line deflections will remain constant for a given lateral load for pile penetrations beyond a critical depth. This critical depth was found to be about 30 ft. for soft clay and about 25 ft. for loose sand with a 16-inch diameter pipe pile. Results of modeling a more flexible 10.75-inch diameter pipe pile in soft clay indicate that the critical depth is no greater than 30 ft. in this case as well. Deflections increase significantly for penetrations less than the critical depth. Generally, the critical depths will be somewhat shallower for stiffer or denser soils.
- (b) Pile Tip Embedment-For the soft clay case, the pile tip was embedded 2 ft. into soft rock ($c = 4,000$ psf, $\gamma = 130$ pcf, $\epsilon_{50} = 0.004$, and $k = 1,000$ pci). Table 1.1.3-2 contains a summary of results. Tip embedment had no effect on a 30 ft. pile, and a very minimal effect on a 15 ft. pile with a fixed-head under a lateral load of 20 kips. The only significant decrease in deflection due to tip embedment occurred with the free-head, 15 ft. pile.

In the analysis performed for design curve development, 15 ft., HP 10x42 piles in saturated loose sand or soft clay with a 2 ft. embedment into soft or weathered rock were considered. At a 6 kip lateral load, deflections were only decreased about 6 percent from the full depth saturated loose sand case. However, compared to the full-depth saturated soft clay case, deflections were decreased about 30 percent at a 6 kip lateral load. For a 30 ft. HP10x42 pile in saturated soft clay, tip embedment had no effect on deflections. Based on these results, pile tip embedment will have the greatest effect on short piles in saturated, soft clay.

- (c) Axial Load-As shown in Table 1.1.3-3 both deflections and moments increase with increasing axial load with constant lateral force of 20 kips. The effect is much more significant for a free-head pile than it is for a fixed-head pile. Results of a similar analysis of a more flexible 10.75-inch diameter pipe pile supports these conclusions.
- (d) Pile Head Fixity-For a free head pile (zero fixity) there is no moment at the top of the pile. A fixed-head pile (100 percent fixity) has a sufficient resisting moment at the top of the pile to ensure no rotation. To model fixities between these limits, a resisting moment equal to the appropriate fraction of the resisting moment developed in the 100 percent fixed condition was applied to the top of the pile. That is, for 25 percent fixity, 25 percent of the resisting moment developed for the fixed-head condition was applied to the top of the pile. The effect of pile head fixity was investigated for a pile driven into soft clay with a 2 ft. tip embedment into soft rock. Deflections versus pile head fixity for a given pile length and lateral load are plotted in Figure 1.1.3-1. For a given pile length, the greater the pile head fixity the less the deflection.

Figure 1.1.3-1 also contains a plot of the absolute value of the maximum moment versus pile head fixity. Using the sign conventions as defined for the COM624G program, positive moments in the pile increase with decreasing pile head fixity and are greatest for a free-head pile. Negative moments in the pile increase with increasing pile head fixity and are greatest for an 100 percent fixed head pile. Therefore, as shown in Figure 1.1.3-1, for fixities between 0 and 100 percent, the net moment in the pile decreases.

Although the magnitude of the differences may vary, the effect of pile head fixity on deflection and moment as illustrated in Figure 1.1.3-1 will hold true for any pile/soil combination.

- (e) Groundwater - As shown in Table 1.1.3-1, the location of the groundwater table may have a significant effect on lateral deflections. The influence of the water table is most significant for the free-head pile in sand.

Table 1.1.3-1 - Parametric Studies-Groundwater

Lateral Load (kips)	Groundline Deflection (in.)							
	Sand				Clay			
	Fixed		Free		Fixed		Free	
	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Wet
5	0.06	0.06	0.17	0.18	0.01	0.01	0.04	0.04
10	0.13	0.13	0.35	0.47	0.04	0.04	0.16	0.17
20	0.26	0.32	0.78	1.24	0.16	0.17	0.71	0.76

NOTES:

1. Pile - 16 inch pipe
2. Pile Length - 30 ft.

Table 1.1.3-2 - Parametric Studies-Pile Tip Embedment

Pile Length (ft.)	Lateral Load (kips)	Groundline Deflection (in.)			
		Free Head		Fixed Head	
		w/ Embed.	w/o Embed.	w/ Embed.	w/o Embed.
15	10	0.99	1.9	0.12	0.12
	20	>>	>>	0.42	0.44
30	10	0.35	0.35	0.09	0.09
	20	1.3	1.3	0.32	0.32

NOTES:

1. >> - Excessive groundline deflections indicate soil failure
2. Pile: 16 inch pipe
3. Soil Condition: Soft Clay

Table 1.1.3-3 - Parametric Studies-Axial Load

Axial Load (kips)	Groundline Deflection (in.) / Max. Moment (in.-kips)	
	Free Head	Fixed Head
0	1.23 / 118	0.32 / -122
6	1.42 / 133	0.33 / -125
12	1.66 / 150	0.38 / -127

NOTES:

1. Pile - 16 inch
2. Pile Length - 30 ft.
3. Soil Conditions - Soft Clay
4. Lateral Load - 20 kips

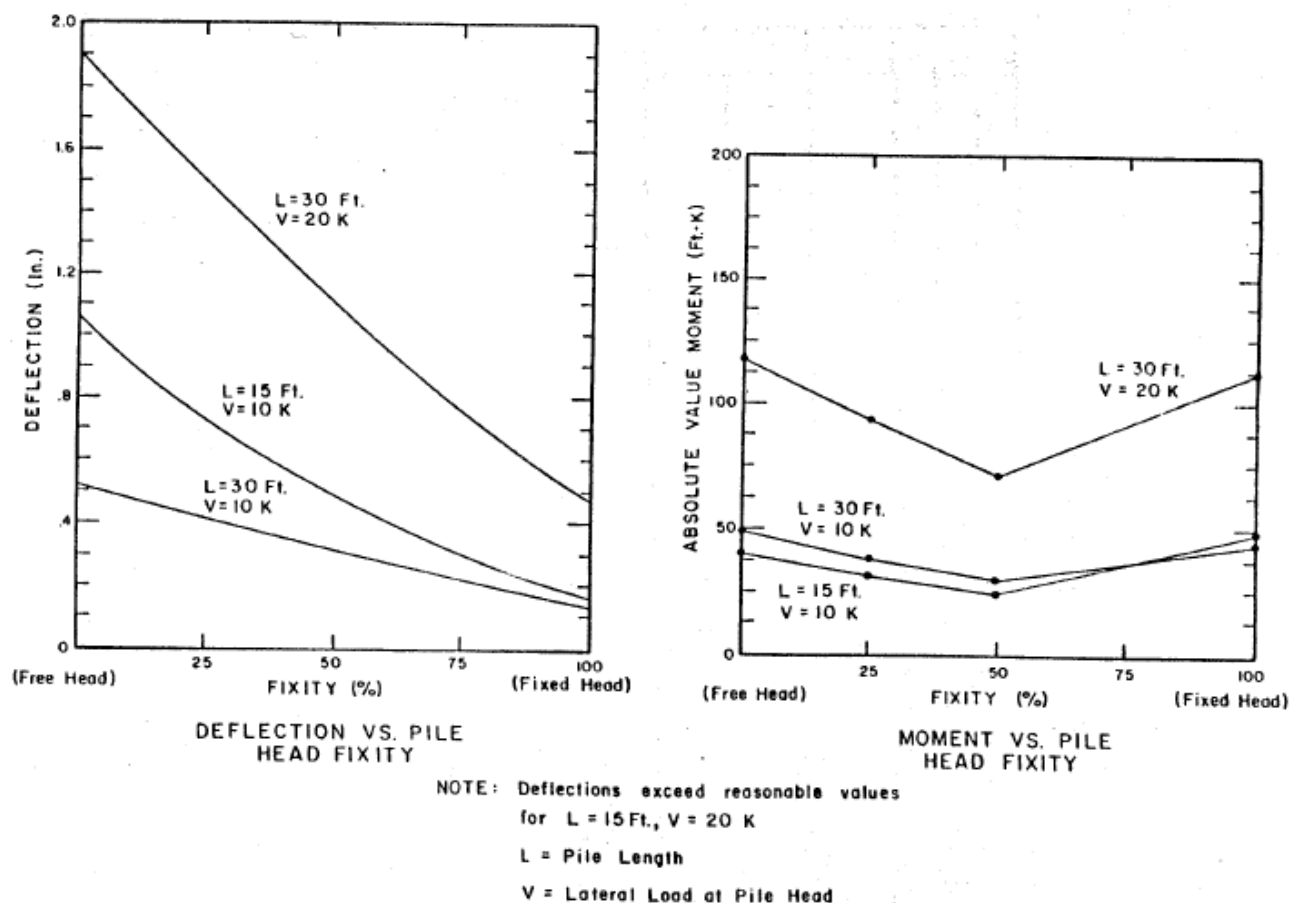


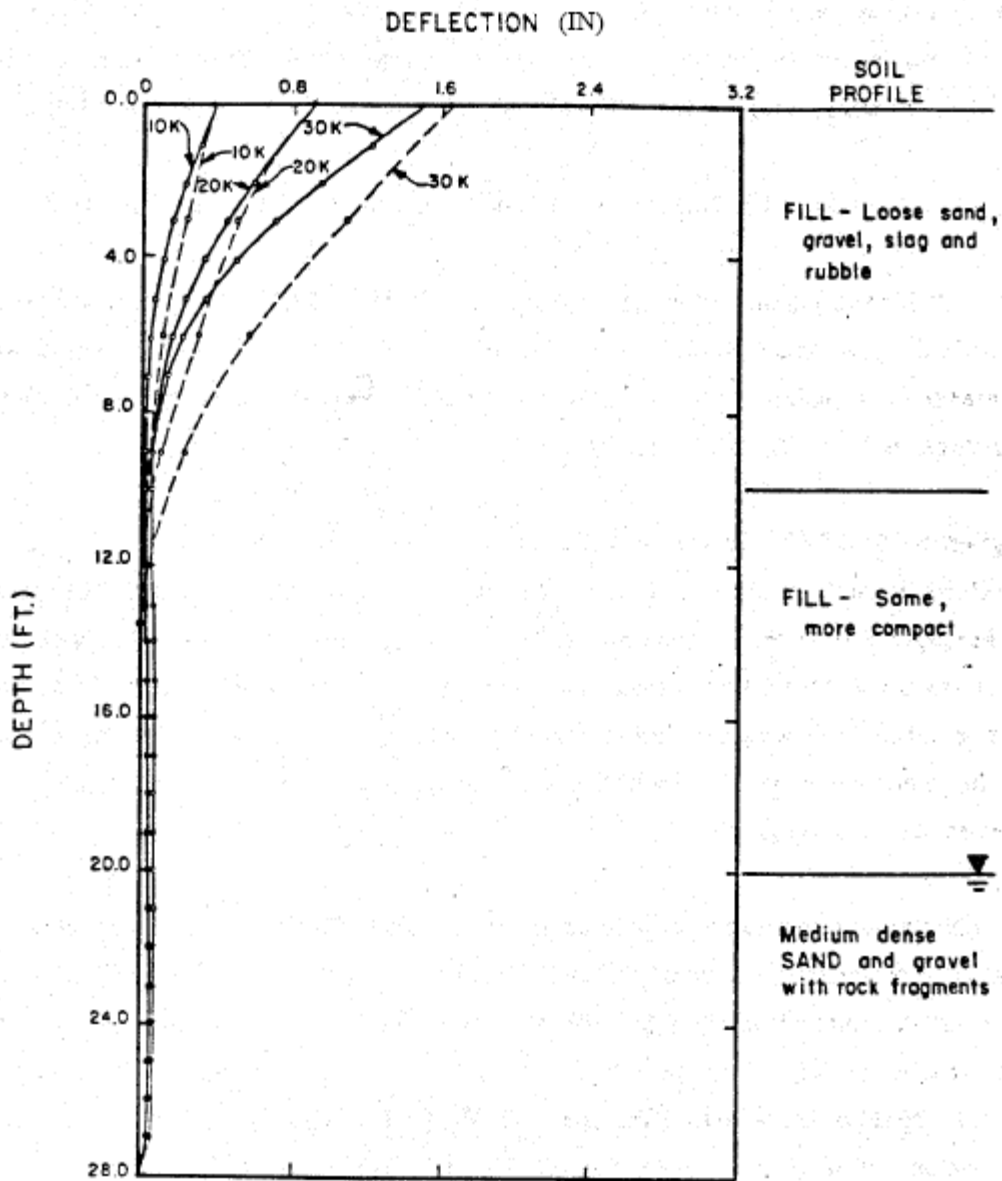
Figure 1.1.3-1 - Head Fixity

1.1.4 Comparison of Computed Results with Load Test Results

A load test program, sponsored by PennDOT, was conducted for the design of LR 1021, LR 1026, and LR 1040 in the Pittsburgh area [7]. The deflected shape, as indicated by inclinometer data, of laterally loaded test piles from two sites has been compared with the deflected shape computed by the COM624G program.

Figure 1.1.4-1 shows actual and computed deflection profiles for a 60 ft., HP 12x74 pile. The soil profile in the vicinity of the load tested pile consisted of 10 ft. of a loose sand, gravel, slag, and rubble fill; 10 ft. of more compact fill with similar constituents; and 40 ft. of natural, medium dense sand and gravel with rock fragments. These layers were input into the COM624G program as loose sand, medium dense sand, and saturated medium dense sand. Although the computed deflected shape of the pile shows greater deflection along the length of the pile, the agreement between ground line deflections is quite good.

At the other site, the soil profile consisted of 3 ft. of soft silty clay, 15 ft. of stiff silty clay, and 7 ft. of weathered and broken shale. Although the materials had a significant silt content, the strata were all modeled as clays ($\phi = 0$) and the water table was assumed to be at the top of the weathered rock. The load tested pile was an HP 10x42, approximately 25 ft. in length. Figure 1.1.4-2 shows actual and computed deflection profiles for the test pile. It is evident from the plots that using the p-y curves for clay for the conditions at this site results in significantly overestimating deflections at high loads and underestimating deflections at low loads. The treatment of silty soils is addressed in Section 1.1.6.

**LEGEND:**

- Deflection Profile Obtained By Inclinator For Pile No. 13, Site No. 2 L.R. 1021, L.R. 1026, L.R. 1040 Pile Load Test Program.
- Deflection Profile Generated By Computer Program, COM 624 G

PILE DATA:

HP 12 x 74, 60 ft. Length

Figure 1.1.4-1 - Comparison of Load Test Results with Computer Generated Deflection Profile

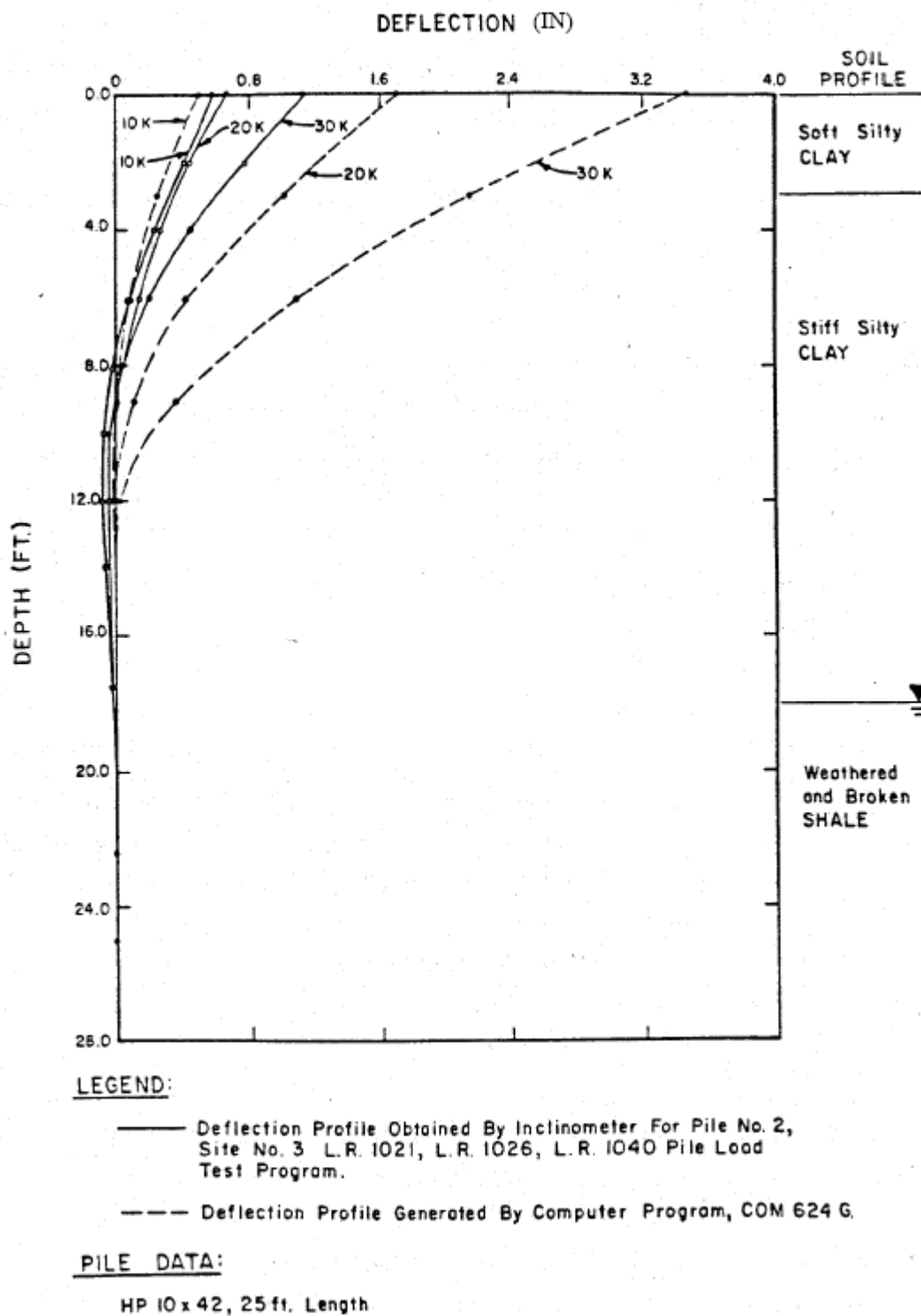


Figure 1.1.4-2 - Comparison of Load Test Results with Computer Generated Deflection Profile

1.1.5 Design Curve Development

Based on the results of analyses using COM624G, design curves presented in 1.1.8 have been developed. The curves cover five pile types for a range of lengths, soil conditions, and loads.

The pile types used for development of design curves are summarized in Table 1.1.5-1. Soil conditions considered are summarized in Figure 1.1.6-1. For a discussion of the use of the given soil profiles to model actual conditions, see 1.1.6.

The parametric studies indicate that deflections and moments will increase with increasing axial loads. A relatively high axial load has been applied to the piles studied to provide conservative results. Axial loads corresponding to the following stresses have been utilized for the various pile types:

- (a) Steel H-piles and pipe piles - 12 ksi
- (b) Cast-in-place piles - $0.35 f'_c$
- (c) Concrete piles - $0.30 f'_c$
- (d) Timber piles - 0.60 ksi

With axial loads higher than these, the moments will be somewhat unconservative due to the additional moment caused by the displacement of the vertical load by the deflection. Investigation of the program results showed that the actual moment increases approximately 10 percent when the pile is stressed to its ultimate axial structural capacity. However, at this level, the capacity is primarily controlled by the axial stress, and an increase of 10 percent in the moment results in an overstress of less than 2 percent in the pile. Therefore, the additional moment can be neglected in most cases. Exceptions include high strength steels and concrete, in which case the design moment shown on the curves shall be increased by an amount equal to the additional axial load times the deflection.

The cases described were analyzed for lateral loads of up to 30 kips. In a bridge, live loads would be considered cyclic. Therefore, the design charts have been developed for cyclic loading conditions. This is likely to be conservative, particularly for abutments, because the total lateral load would have both static and cyclic components. The methods for developing p-y curves utilized in the COM624G program are based on load tests for both static and cyclic loads. In general, effects of cyclic loading, as modeled by the COM624G program, increase with increasing load.

Pile deflections and bending moments from lateral loads are a function of the pile fixity and group effects. Pile fixity is the amount that the pile head is restrained against rotation. When the pile head is completely restrained, the pile is a fixed-head pile (100 percent fixity). When the pile head is completely free to rotate, the pile is a free-head pile (0 percent fixity). Neither condition is typically met in abutments and piers. Instead, the fixity varies due to several factors.

- (a) Pile Cap Stiffness - The pile cap and the piles act as a frame when subject to lateral loads. The cap restrains the rotation of the pile head. The amount of restraint is proportional to the relative stiffness between the cap and the piles. A thicker cap or an additional row of piles would provide greater fixity. This frame action is not available when a single row of piles is used and restraint is only provided by the surrounding ground. Therefore, the fixity of a single row of piles is closer to zero.

Table 1.1.5 -1- Lateral Pile Loading Pile Types

Pile Type	Size of Nominal Diameter		Moment of Inertia (in. ⁴)	Modulus of Elasticity (psi)	Remarks
Steel H-Pile	10x42		71.7	29x10 ⁶	weak axis
	10x42		210		strong axis
	10x57		294		strong axis
	12x53		393		strong axis
	12x74		569		strong axis
	14x117		1,220		strong axis
Steel Pipe	10.75 in		161	29x10 ⁶	Structural capacity from steel only assumed.
	16 in		562		
	24 in		1,940		
Cast-in-Place	10 in		829	3.27x10 ⁶	Structural capacity from steel shell and concrete only, no reinforcing included; Class A concrete, 28-day strength = 3,300 psi.
	12 in		1,253		
	14 in		2,111		
	18 in		7,390		
Precast or Prestressed Concrete				4.03x10 ⁶	No reinforcing included in structural capacity; concrete 28-day strength = 5,000 psi.
Square	10 in		835		
	12 in		1,728		
	14 in		3,199		
	18 in		8,748		
Octagonal	10 in		155		
	12 in		1,285		
	14 in		2,107		
	18 in		5,751		
Timber	Dia. (1)	Len.	Varies along length of pile due to taper.	1.10x10 ⁶	Used Characteristics conservative for several species of wood (ASTM D 2555).
	8 in 13 in	20 ft			
	10 in 15 in	40 ft			
	12 in 18 in	60 ft			

NOTE: Diameter 3 ft. below butt as per ASTM D 25

Table 1.1.5-2 - Comparison of Computer Generated Pile Capacities with Load Test Results
Based on Fixity and Group Effects

Deflection (in.)	0.2	0.4	0.6	0.8	1.0
Group Efficiency-Load Test ⁽¹⁾	1.9	1.6	1.4	1.3	1.2
50 Percent Fixed/Free (Avg.) ⁽²⁾	1.3	1.3	1.3	1.3	1.3
100 Percent Fixed/Free (Avg.) ⁽²⁾	2.4	2.4	2.4	2.4	2.4
NOTES: 1. Ref. [8]; Group efficiency = Ratio of individual pile capacity in a group to the capacity of a single pile, for a given deflection; Group considered had pile spacing of 3D, with 4 in of soil removed below the pile cap. Data from full scale lateral load tests. 2. Average value for HP 10x42 and HP 14x117 piles, weak axis, soil profiles 2 and 4; pile capacities for given deflections. Data from COM624G Pile Analysis Program.					

- (b) **Pile/Pile Cap Connection** - In order for the cap to restrain the pile head, the pile shall be securely fastened to the cap. According to Davisson [9], with structural details commonly used a fixity of 50 percent is obtainable. Reinforcement located at the base of the pile cap was not in common use at the time this statement was made. Any cracking at the base of the cap would reduce the fixity of the connection. With a 1 ft. pile embedment and cap reinforcement located at the bottom of the cap, cracking at the base of the cap will be minimized [10]. It is likely that fixities greater than 50 percent will be realized.
- (c) **Lateral Loads** - Under small lateral loads, little rotation of the pile takes place and the fixity is approximately 100 percent. As the lateral loads increase, rotation of the cap as well as some movement of the pile in the pile/pile cap connection take place, allowing the pile head to rotate and decreasing the fixity.

Frame action in pile groups will cause a decrease in total deflection due to an increase in pile head fixity. However, given a constant pile head fixity, a group of piles would be likely to deflect more than a single pile due to disturbance of the surrounding soils by movement of adjacent piles. Full scale load test data for pile groups is minimal. In general, results of full scale and model load tests [9,11,12,13] indicate that deflection of a group is greater than deflection of a single pile. Interpretation of such data is complicated, however, by the difficulty in determining and controlling pile head fixity for single piles and pile groups.

Group effects from adjacent piles are more significant in the direction of loading. Based on model tests by Prakash [9], and confirmed by model tests by Cox [11], there will be no group effects for spacings greater than about 8D (where D = pile diameter) in the direction of loading or for spacings greater than about 2.5D normal to the direction of loading. For most abutments and piers, spacing in the direction of loading will be less than 8D.

Full scale lateral load tests in Pennsylvania as reported by Kim [14,15,16] indicate that a group of piles will be more efficient for deflections than a single pile. Kim's tests included comparing the behavior of a single vertical HP 10x42 pile to that of two groups of vertical HP 10x42, one spaced at 4D, the other at 3D. Both the single pile and the pile groups were embedded 12 in. into a 4 ft. thick pile cap. All pile caps had reinforcing at the base. Axial stresses of about 6 ksi were applied during lateral load testing.

The results of Kim's study indicate that pile group efficiencies (the ratio of individual pile capacity in a group to the capacity of a single pile, for a given deflection) were greater than 1.0 for all groups. Group efficiencies were greater for greater pile spacings. It is probable that these trends are due to the greater fixity of the pile head for pile groups. That is, the decrease in deflection due to the increased fixity of the pile head (frame action) more than compensates for the increase in deflection due to group effects from adjacent piles.

In order to take into account fixity and group effects typical to abutments and piers, a 50 percent fixity was assumed when using the COM624G program to determine deflections. This provides deflections which are conservative for any fixity greater than 50 percent. These fixities can usually be achieved when more than one row of piles is embedded into and connected by a concrete pile cap reinforced at the bottom. A comparison of the program output with group efficiency studies by Kim [14,15,16] was performed. Kim's pile group efficiencies were compared to the ratio of pile capacity of a free-head pile to the capacity of a pile with 50 percent fixity. As shown in Table 1.1.5-2, the results indicate that for 50 percent fixity the program produces reasonable deflections for pile groups typical to abutments and piers.

As shown in Figure 1.1.3-1, using 50 percent fixity will minimize the maximum moment in the pile in some cases. To take into account the potential for higher fixities, 80 percent of the fixed-head maximum moment is shown on the design curves. This moment will be conservative for fixities between 50 percent and 80 percent, and in many cases for fixities somewhat less than 50 percent. Fixities greater than 80 percent would be associated with small lateral loads, where moment is not critical. Fixities less than 50 percent may occur when lateral loads are very high or where there is only a single row of piles.

A pile head fixity less than 50 percent might also occur if reinforcement is not placed at the bottom of the pile cap and lateral loads are high enough to induce cracking. Where piles are closely spaced, placement of a reinforcement cage below the top of the piles may be impossible due to pile misalignment during driving. Hand work would be required to place reinforcing below the top of the piles. The data presented in Table 1.1.5-2 indicate that the ratio of the lateral resistance of a pile with 50 percent fixity to a free-head pile is 1.3 for any given deflection. Additional analysis indicates that for a given moment 1.3 is appropriate for the average ratio of the lateral resistance of a pile with 80 percent fixity to a pile with 20 percent fixity. Even with a cracked pile cap, at least 20 percent head fixity should be retained. Therefore, if the actual lateral load is less than 75 percent of the lateral resistance as determined by the design curves, reinforcing is not required to be located below the top of the piles. Alternatively, if reinforcement cannot economically be placed below the top of the piles, no more than 75 percent of the lateral resistance as determined by the design curves shall be used.

1.1.6 Soil Conditions

Design curves have been developed for the seven soil profiles shown in Figure 1.1.6-1. In the descriptions of these profiles all granular material is referred to as sand and all cohesive material is referred to as clay. Actual soil conditions may be more complex than the simplified soil profiles presented. Engineering judgment shall be employed when selecting the appropriate soils profile to model actual conditions. Soils from the ground surface to a depth equal to five to ten pile diameters have the greatest influence on laterally loaded pile behavior [5].

Soil Profiles 1 through 4 should cover most soil conditions if the soils within a depth of five to ten pile diameters are assumed to control the behavior of a laterally loaded pile. As examples, lateral load resistance can be established for the load test piles described in D10.7.3.8.

Referring to Figure 1.1.4-1, this 60 ft. long HP 12x74 is embedded in granular material. The top 10 ft. is a loose sand above the water table. Using Soil Profile 1 should give a conservative estimate of lateral load resistance. Using Figure 1.1.8-1, for a deflection of 0.5 in., the lateral load resistance would be 12 kips. The results of the lateral load test are shown on Figure 1.1.4-1. The load test was performed on a free-head pile, therefore, deflections would be greater than those expected for a pile with at least 50 percent head fixity gained by pile cap embedment. The load test resulted in approximately 0.4 in. of deflection for a 10 kip load. The recommended lateral load of 12 kips, as expected, seems slightly conservative but reasonable.

The soil profile for the other load test considered is shown on Figure 1.1.4-2. The 25 ft long HP 10x42 is embedded in predominantly cohesive soils. Surface soils consist of 3 ft. of soft silty clay overlying 15.0 ft. of stiff silty clay. Both strata are above the water table. These soil conditions fall between Soil Profile 3 and Soil Profile 4. Using Soil Profile 3 would be very conservative as it models a full-depth, soft, saturated clay. From Figure 1.1.8-1, the lateral load resistance for Soil Profile 3 is 7.5 kips. Soil Profile 4 is much closer to actual conditions, with the exception of the top 3 ft. of soft material. From Figure 1.1.8-3, the lateral load resistance for Soil Profile 4 is 16.5 kips. A conservative estimate for the actual soil conditions can be obtained by using a value halfway between the values obtained for Soil Profile 3 and 4. Therefore, for a lateral load resistance of 12 kips would be recommended. This value is reasonable when compared to the deflections obtained from the load test of a free-head pile, i.e., 0.58 in. for a lateral load of 10 kips and 0.66 in. for a lateral load of 20 kips.

Soil Profiles 5, 6, and 7 are more specialized than the first four. Profiles 5 and 6 were developed to model very poor surface soil conditions. Any greater depth of very loose or very soft material shall be analyzed independently. Soil Profile 7 models tip embedment. Parametric studies showed that pile tip embedment into weathered or soft rock has no effect on piles 30 ft. in length. Soil profiles for full-depth sand and clay can be used for depths of 30 ft. or more. The effect on 15 ft. piles was most significant with saturated soft clay. Therefore, Soil Profile 7 covers this condition. Because of the minimal effect of tip embedment on piles in saturated loose sand, Soil Profile 1 can be used to adequately model this case.

Many soils in Pennsylvania exhibit a high silt content. Methods for determining p-y curves are limited to sands and clays. No model has yet been developed to simulate a soil exhibiting both a cohesion, c , and an angle of internal friction, ϕ , value. Therefore, a judgment must be made as to whether the soils are predominantly granular or predominantly cohesive. The comparison of computed results to load test results (A10.7.3.12 and D10.7.3.12) indicates that for clays, the p-y curve analysis tends to underestimate groundline deflections at low loads and overestimate groundline deflections at higher loads. Agreement of groundline deflections for sands was quite good. So that groundline deflections are not underestimated for clays at low loads for Soil Profiles 1 through 4, only the more conservative results for sand are shown on the lateral load vs. deflection curves at low loads. This curve, shown as a solid line, shall be used for both sand and clay. At higher loads the curves for clay are shown with a broken line.

An independent analysis with the COM624G program shall be performed when actual soil conditions are worse than those covered by the soil profiles. If soil conditions are better than those covered by the soil profiles, an independent analysis may be performed if a substantial cost savings can be realized by a further increase in the lateral load resistance. In many cases, where soil conditions are good, the lateral load resistance will be controlled by the available moment determined by the interactive equations rather than deflections. The effect of improved soil conditions is generally less for the load vs. maximum moment curves than it is for the load vs. deflection curves.

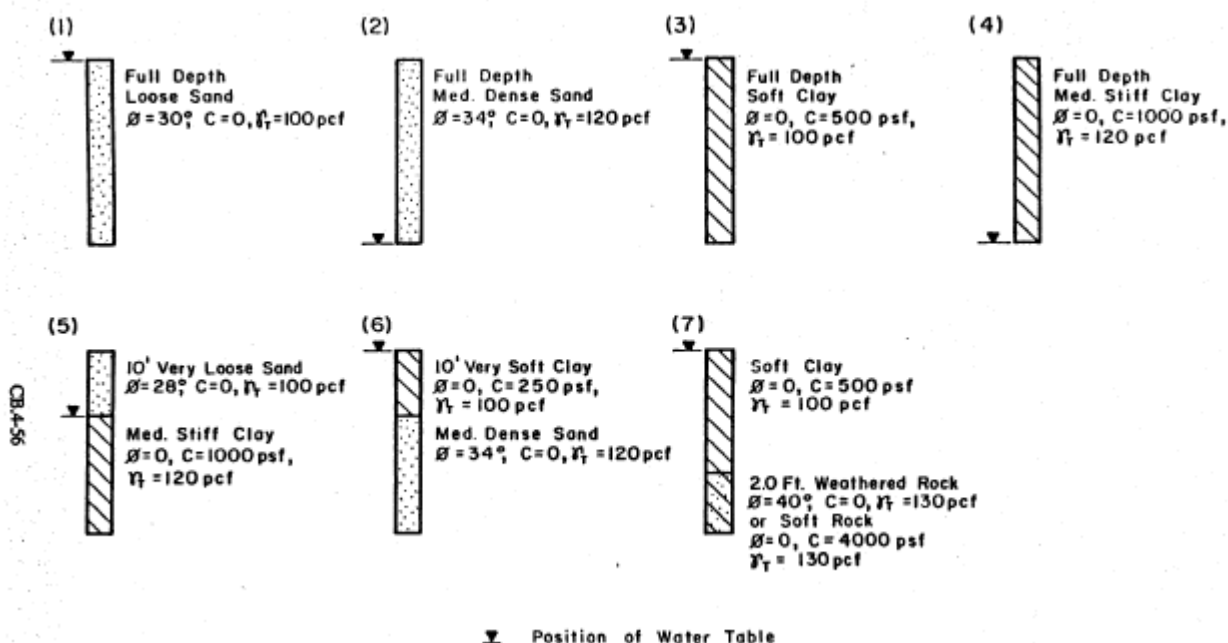


Figure 1.1.6-1 - Lateral Pile Loading Soil Profiles

1.1.7 Short Piles

The deflected shape of a short stiff pile in soft or loose soils indicates that the bottom of the pile is moving towards the load and that little bending in the pile is taking place. The stiffer the pile the more likely it is to act in this manner, given sufficient lateral load. A more flexible pile will bend with the load, and less movement will occur at the base of the pile. The lateral load versus deflection curves (see Figure 1.1.8-8) for Soil Profile 6, 15 ft. piles, illustrate the effect of this behavior. The stiffer HP 14x117 ($I = 1220$ in.⁴) shows greater deflection than the HP 12x74 ($I = 569$ in.⁴) for a lateral load of 8 kips. Therefore, for short piles, increasing the size of the pile may not produce greater lateral resistance based on deflections.

For very short piles, less than 15 ft. in depth, recommended lateral loads resistance are given in Tables 1.1.7-1, 1.1.7-2 and 1.1.7-3. Piles this short are likely to be end-bearing. Timber and precast piles are generally used as friction piles. Precast piles can be used for end-bearing if an H-pile section is cast in the tip. However, this would be uneconomical for such short piles. Therefore, the values in the above-referenced Tables apply to H-piles, pipe piles, and cast-in-place piles only.

Because of the extremely poor soil conditions in the surface 10 ft. of Soil Profiles 5 and 6, no lateral load shall be allowed for a pile less than 15 ft. long in these materials.

Table 1.1.7-1 - Lateral Loads Resistance for Steel H-Piles less than 15 ft. in Length

Soil Profile	Pile Size	Lateral Load Resistance (kips)	Maximum Moment (kip-ft.)
1, 3, 7	10x42 to 12x74	2	9
1	14x117	0	0
3, 7	14x117	2	9
2, 4	10x42	6	15
2	10x57 to 12x74 14x117	8 10	25 38
4	10x57 to 14x117	6	15

NOTES:

1. Values developed for pile length of 10 ft., deflection at top of pile of about 0.25 in.
2. VALUES APPLY TO BOTH WEAK AND STRONG AXIS OF PILE SIZE SPECIFIED.

Table 1.1.7-2 - Lateral Load Resistance for Pipe Piles less than 15 ft. in Length

Soil Profile	Pile Diameter (in.)	Lateral Load Resistance (kips)	Maximum Moment (kip-ft.)
1, 3	All Sizes	0	0
2	10.75 16 24	6 6 6	14 18 28
4	10.75 16 24	6 8 10	12 20 29
7	10.75, 16, 24	4 6	12 27

NOTE: Values developed for pile length of 10 ft., deflection at top of pile of about 0.25 in

Table 1.1.7-3 - Lateral Load Resistance for Cast-in-Place Piles less than 15 ft. in Length

Soil Profile	Pile Diameter (in.)	Lateral Load Resistance (kips)	Maximum Moment (kip-ft.)
1	10, 12, 14, 18	0 2	0 9
2, 4	10, 12, 14, 18	6 10	15 35
3	10, 12, 14, 18	2	6
7	10, 12, 14, 18	2 4	4 15

NOTE: Values developed for pile length of 10 ft., deflection at top of pile of about 0.25 in.

1.1.8 Using Design Curves

The design curves are a compilation of computed results for several pile types, sizes, and lengths for varying soil conditions. Design curves show deflection at the pile head versus lateral load and lateral load versus maximum moment for a particular soil condition and pile type, size, and length. The lateral load versus deflection curves shall be used with factored loads. The lateral load versus moment curves shall be used with factored lateral loads. In the case of multiple lateral loads for a particular group, a composite group factor shall be used. The maximum moment can be assumed to act within the top 12.0' of the pile.

The lateral load design curves are presented as follows:

- (a) Steel H-Piles – Figures 1.1.8-1 to 1.1.8-11
- (b) Cast-in-Place Piles – Figures 1.1.8-12 to 1.1.8-21
- (c) Pipe Piles – Figures 1.1.8-22 to 1.1.8-30
- (d) Precast Concrete Piles – Figures 1.1.8-31 to 1.1.8-48
- (e) Timber Piles – Figures 1.1.8-49 to 1.1.8-55

It is conservative to interpolate linearly between values of the moment of inertia (I), or pile size and pile length. The designer shall not extrapolate beyond the limits covered by the design curves, as in some cases higher lateral loads would cause soil failure. For piles less than 15 ft. in length use the lateral load resistances given in Tables 1.1.7-1, 1.1.7-2, and 1.1.7-3.

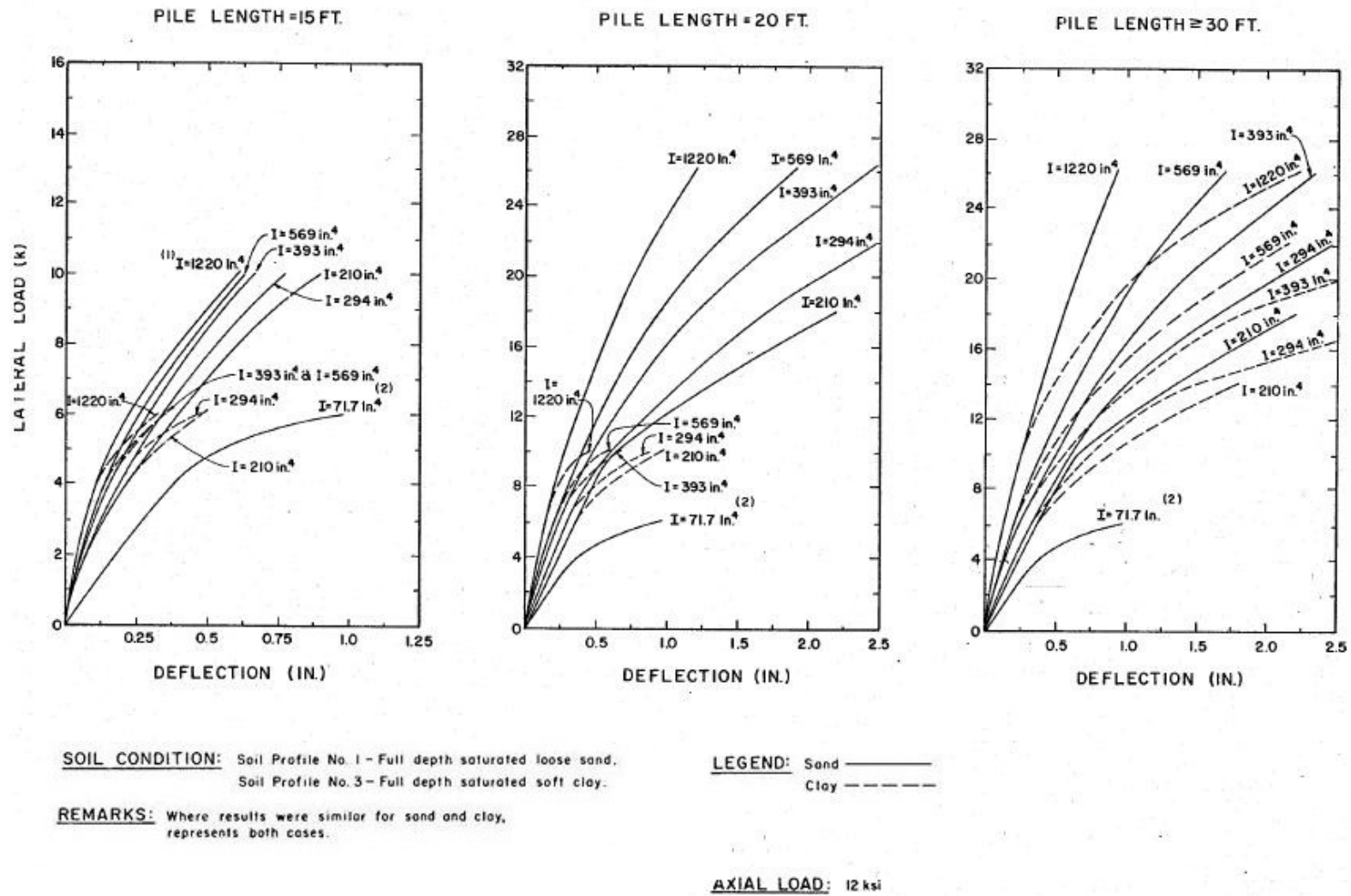


Figure 1.1.8-1 - Load Versus Deflection for Steel H-Pile, Soil Profiles 1 and 3

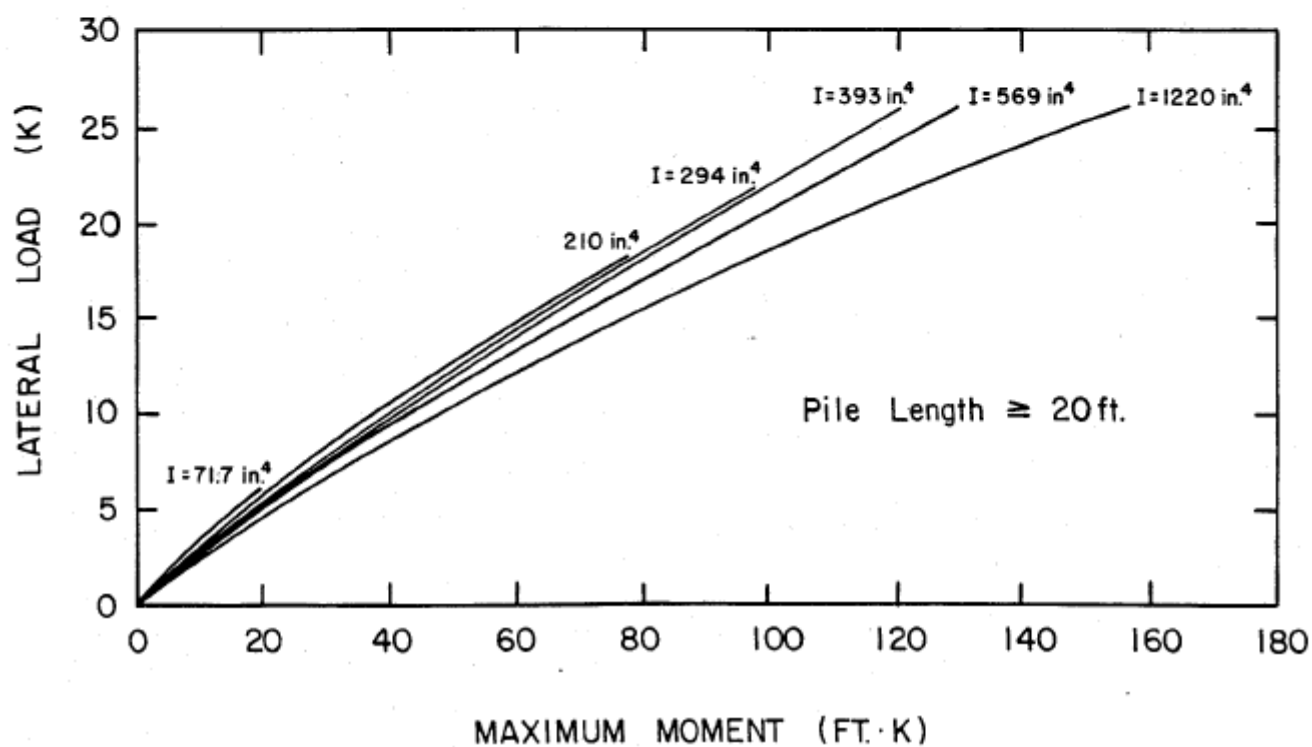
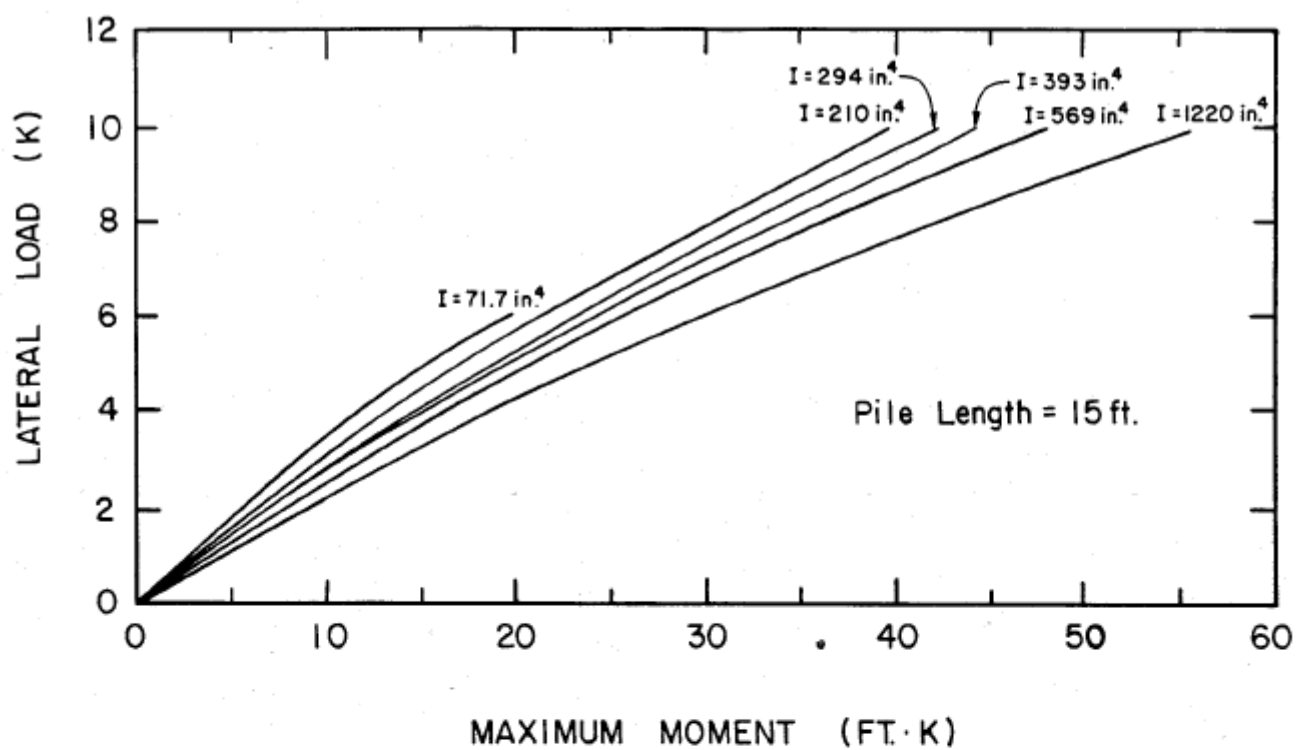


Figure 1.1.8-2 - Load Versus Maximum Moment for Steel H-Piles, Soil Profiles 1 and 3

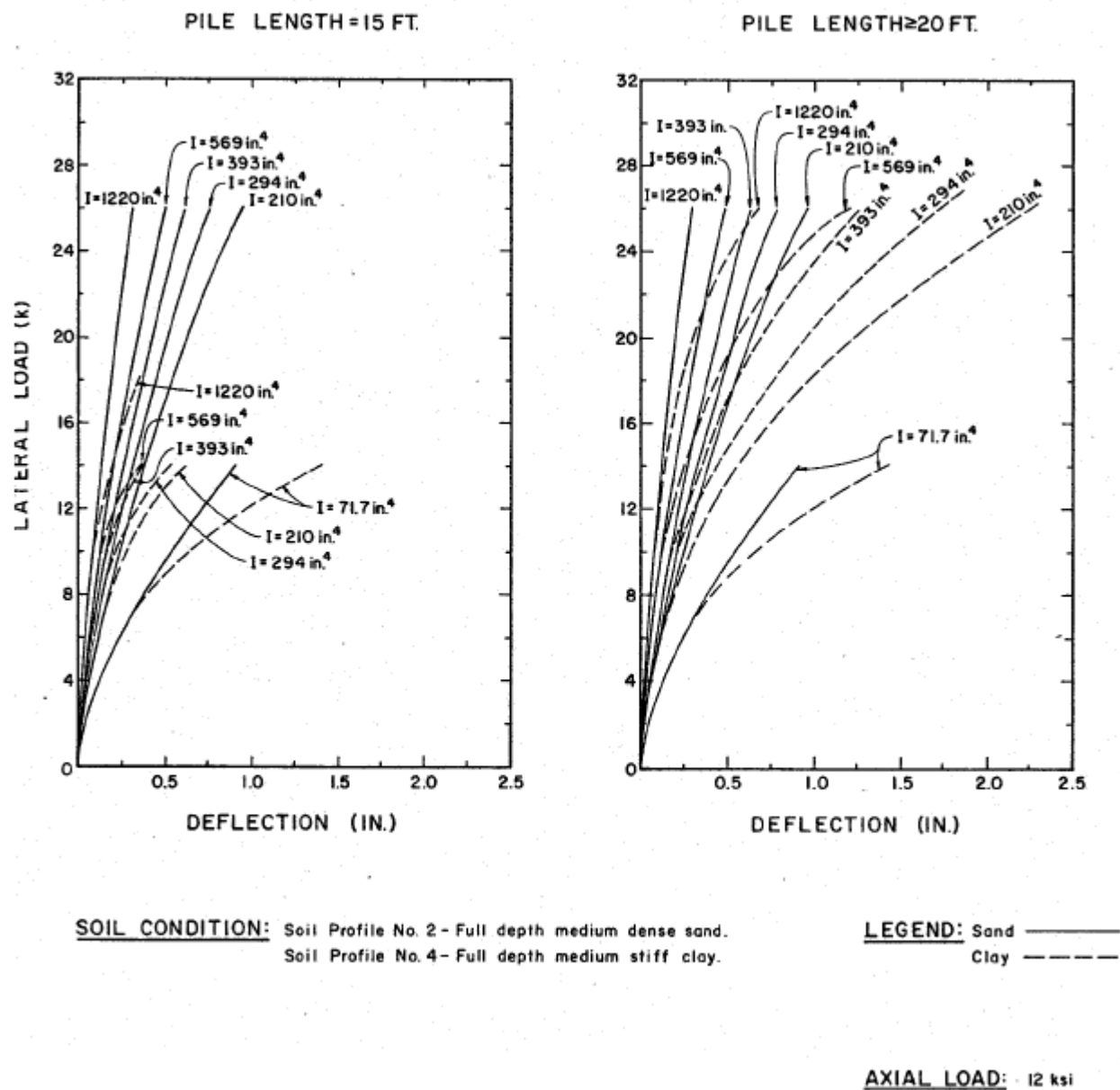


Figure 1.1.8-3 - Load Versus Deflection for Steel H-Pile, Soil Profiles 2 and 4

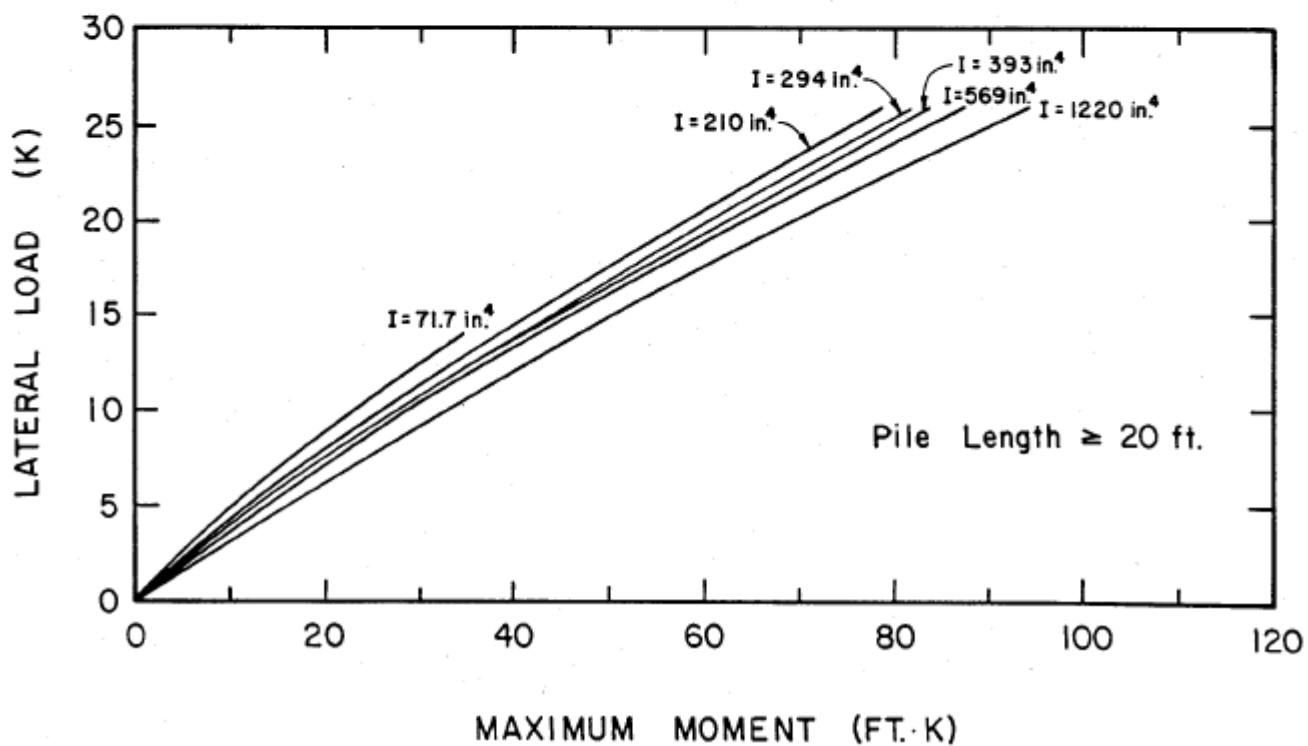
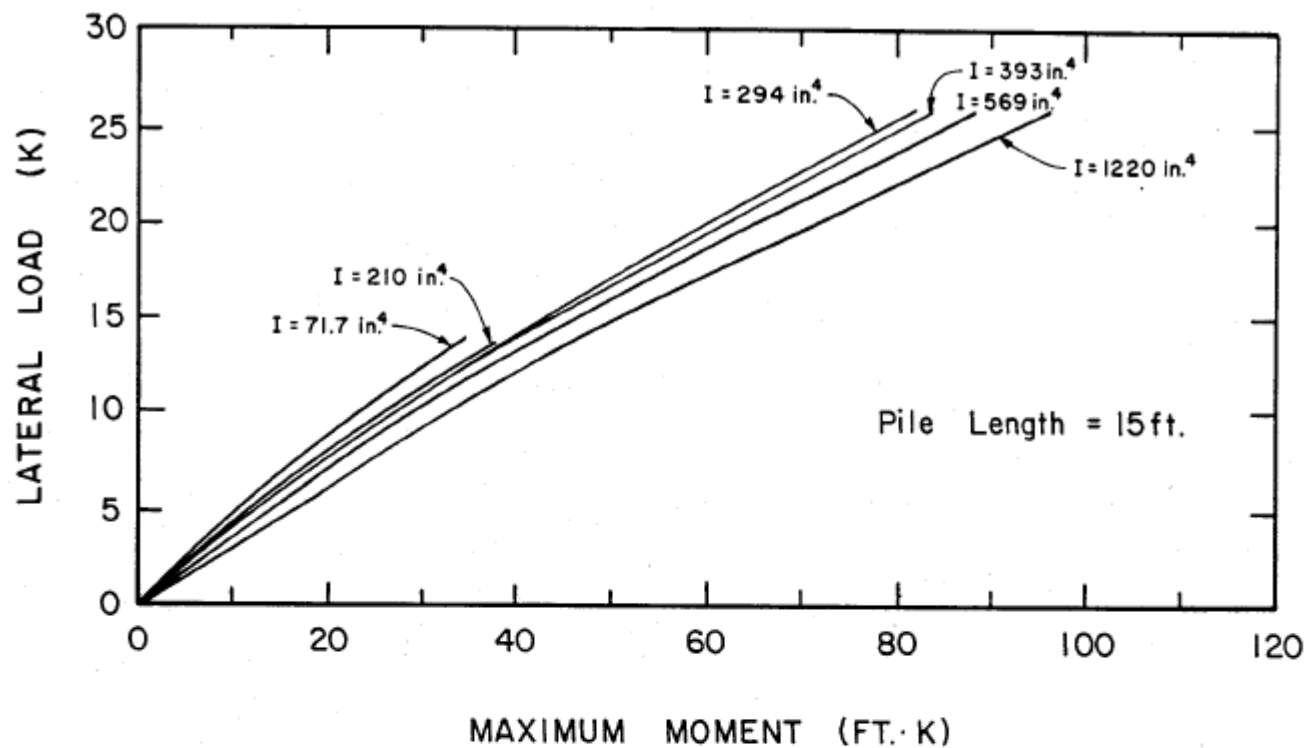


Figure 1.1.8-4 - Load Versus Maximum Moment for Steel, H-Pile, Soil Profile 2

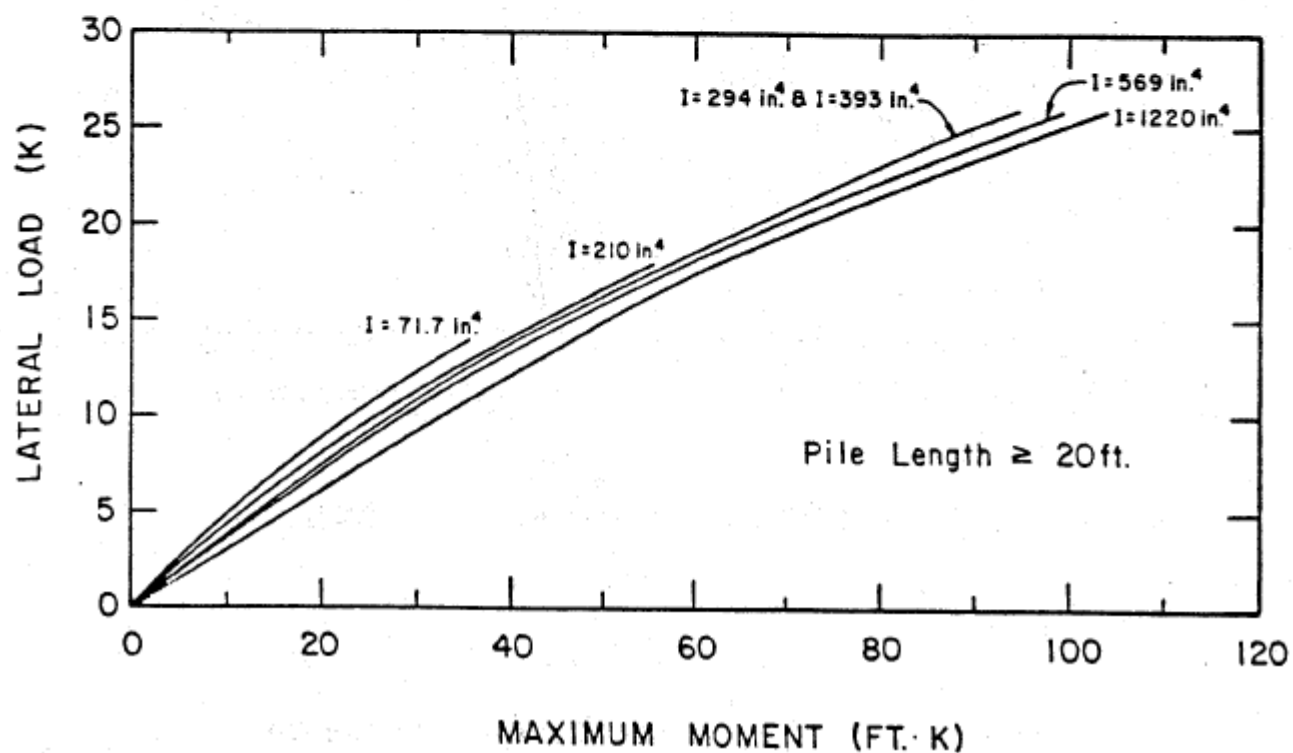
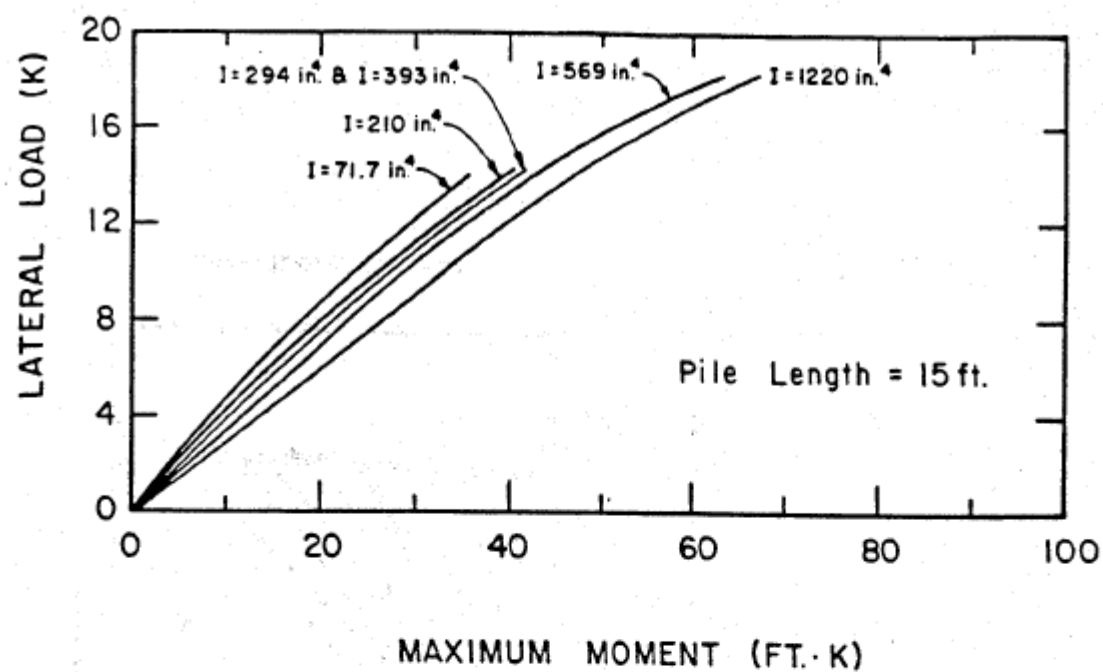


Figure 1.1.8-5 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 4

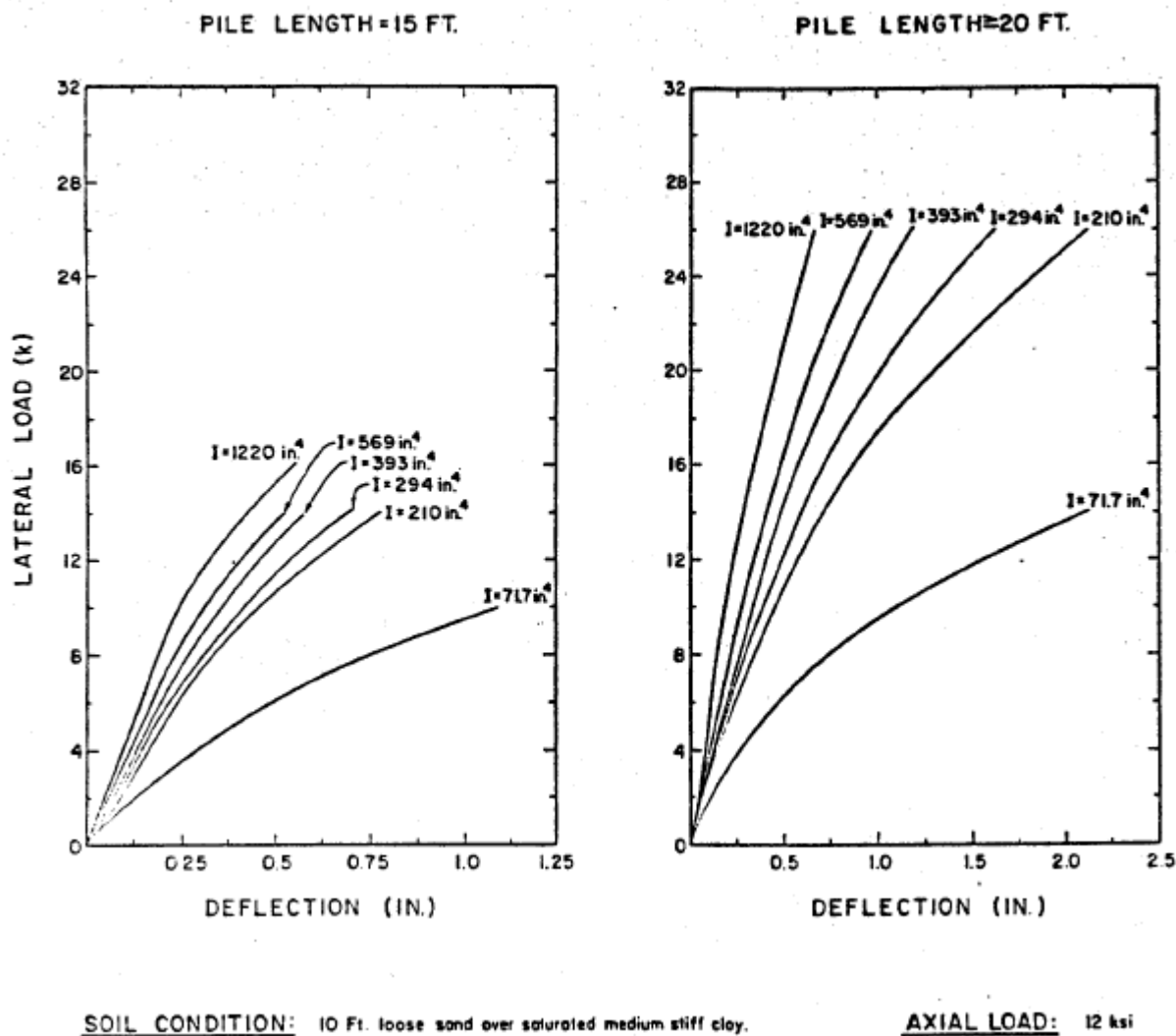


Figure 1.1.8-6 - Load Versus Deflection for Steel H-Pile, Soil Profile 5

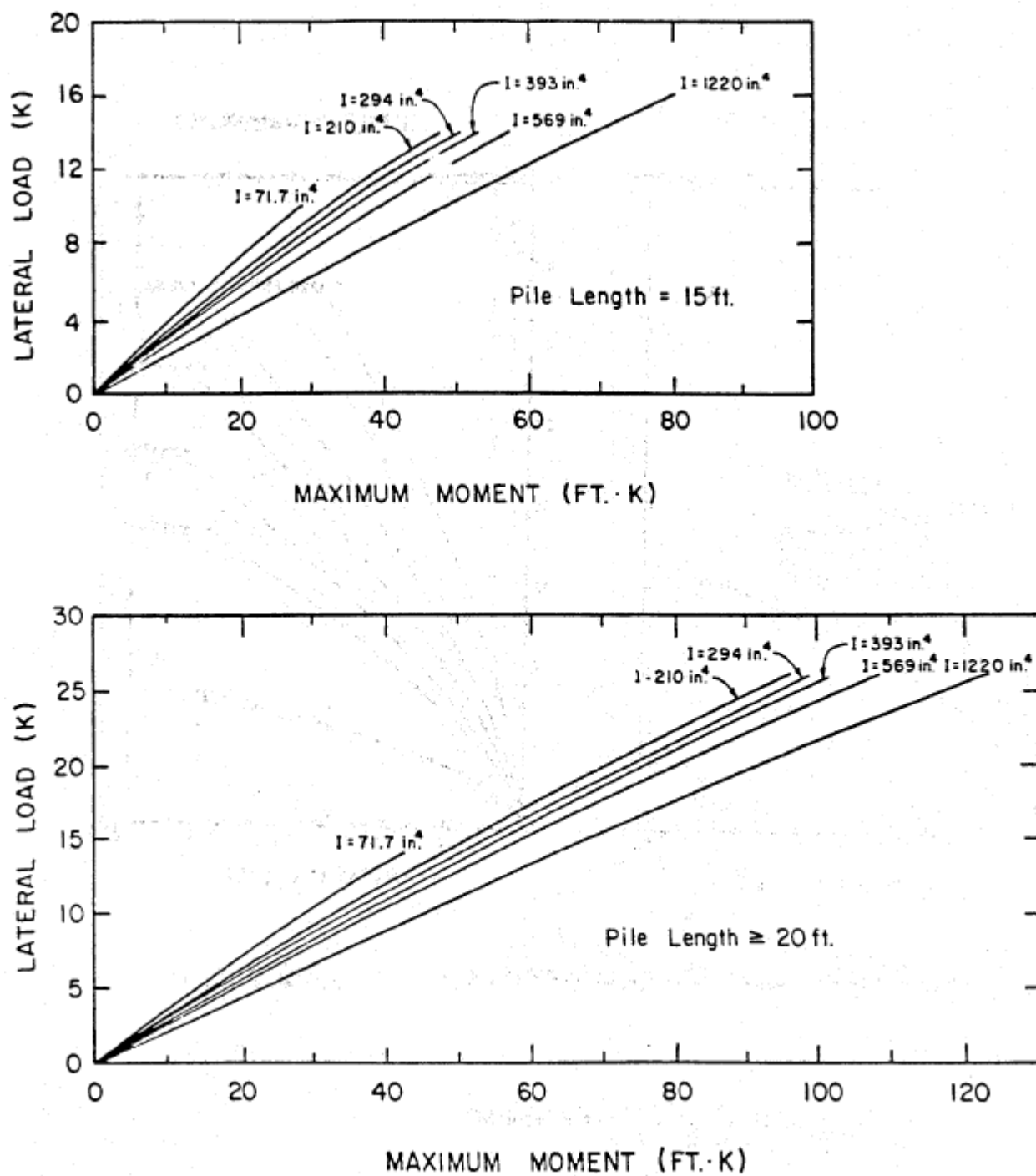


Figure 1.1.8-7 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 5

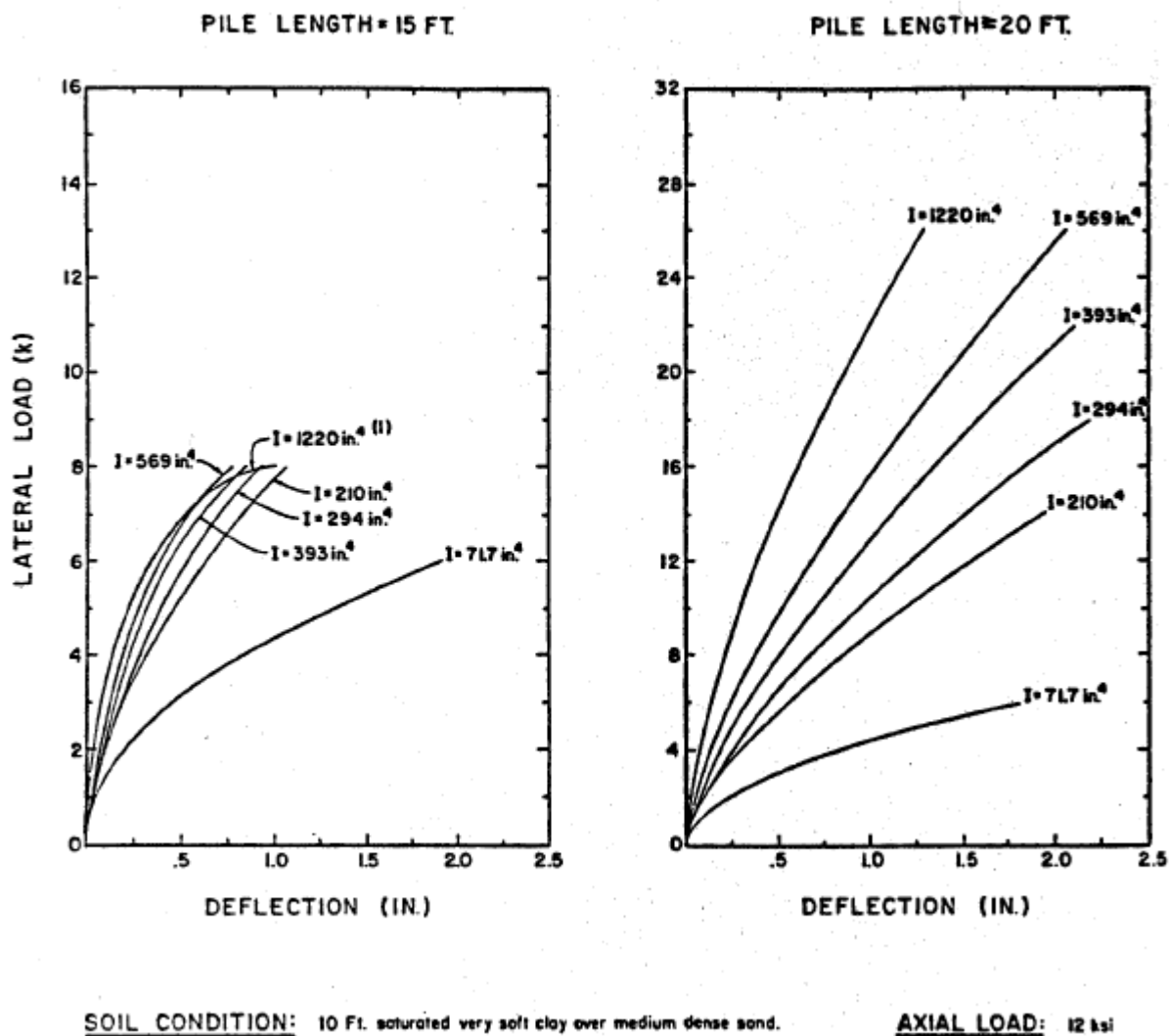


Figure 1.1.8-8 - Load Versus Deflection for Steel H-Pile, Soil Profile 6

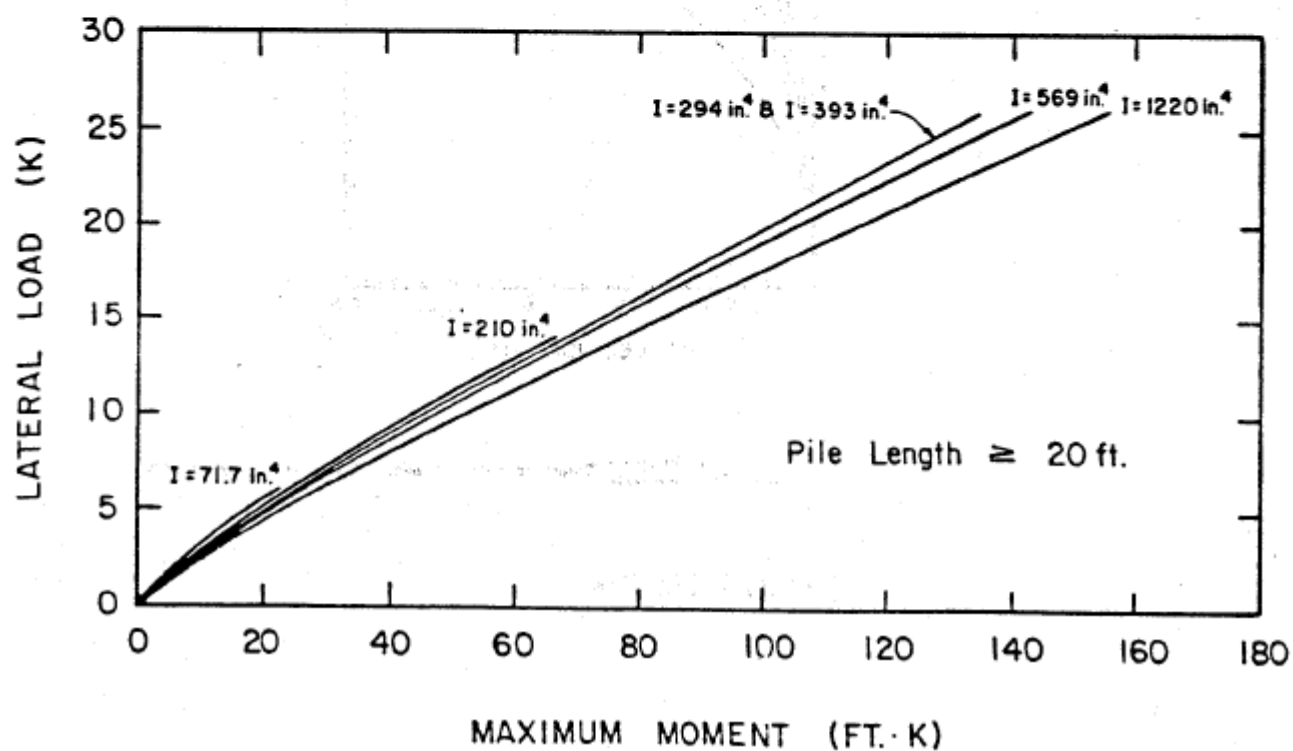
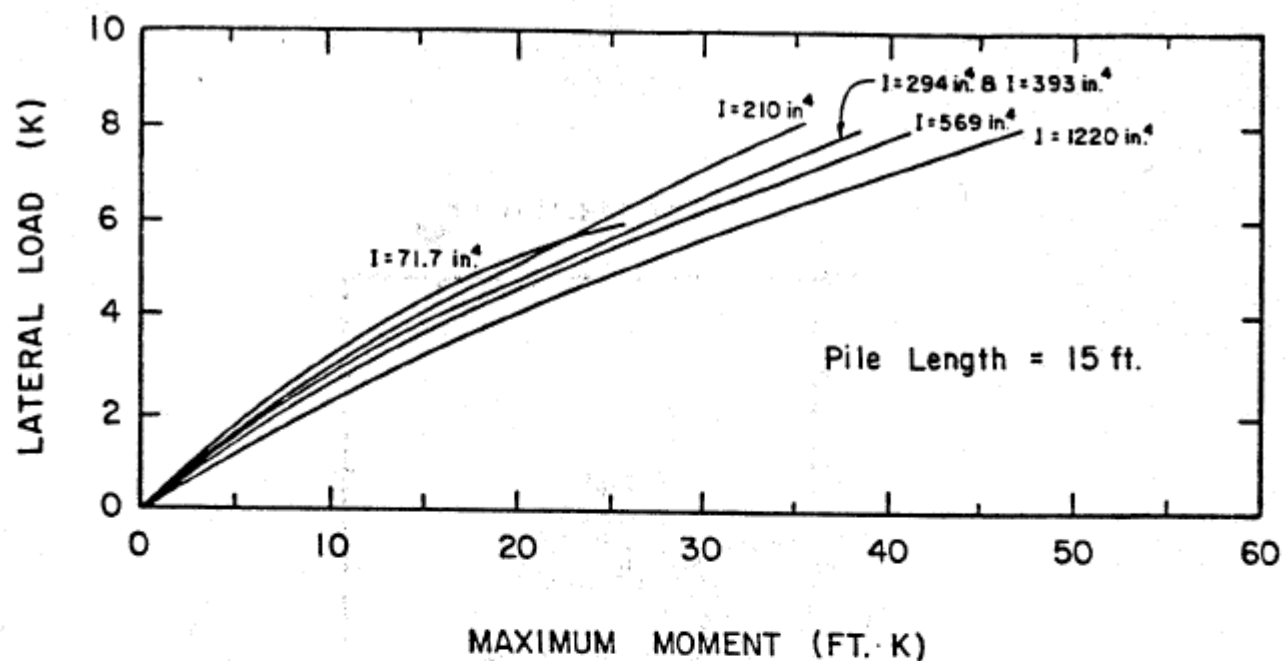


Figure 1.1.8-9 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 6

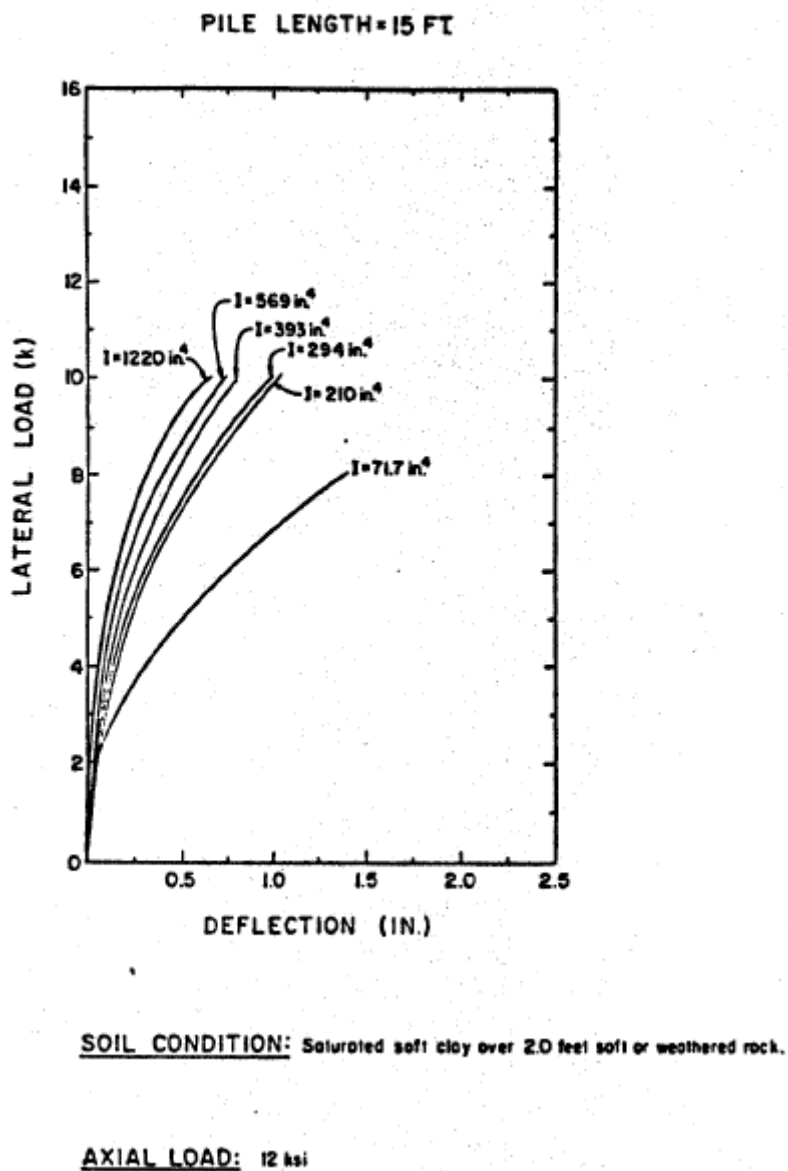


Figure 1.1.8-10 - Load Versus Deflection for Steel H-Pile, Soil Profile 7

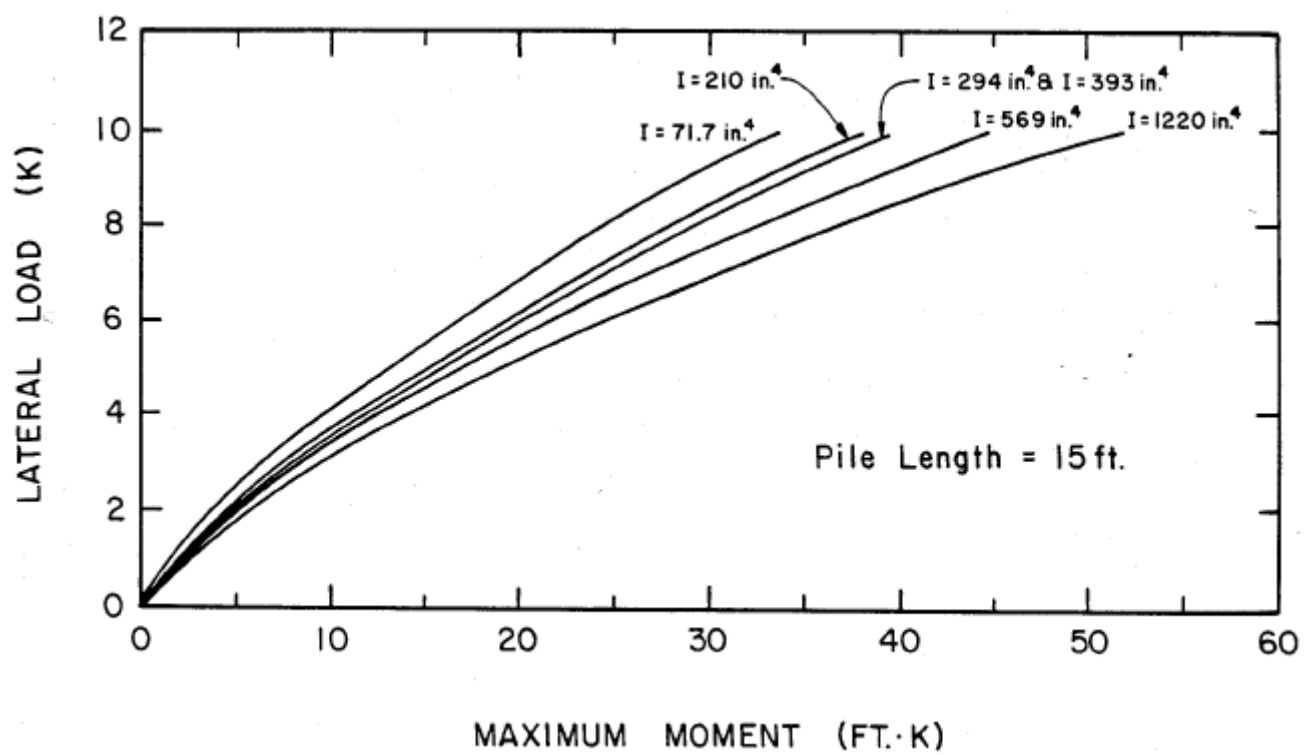


Figure 1.1.8-11 - Load Versus Maximum Moment for Steel H-Pile, Soil Profile 7

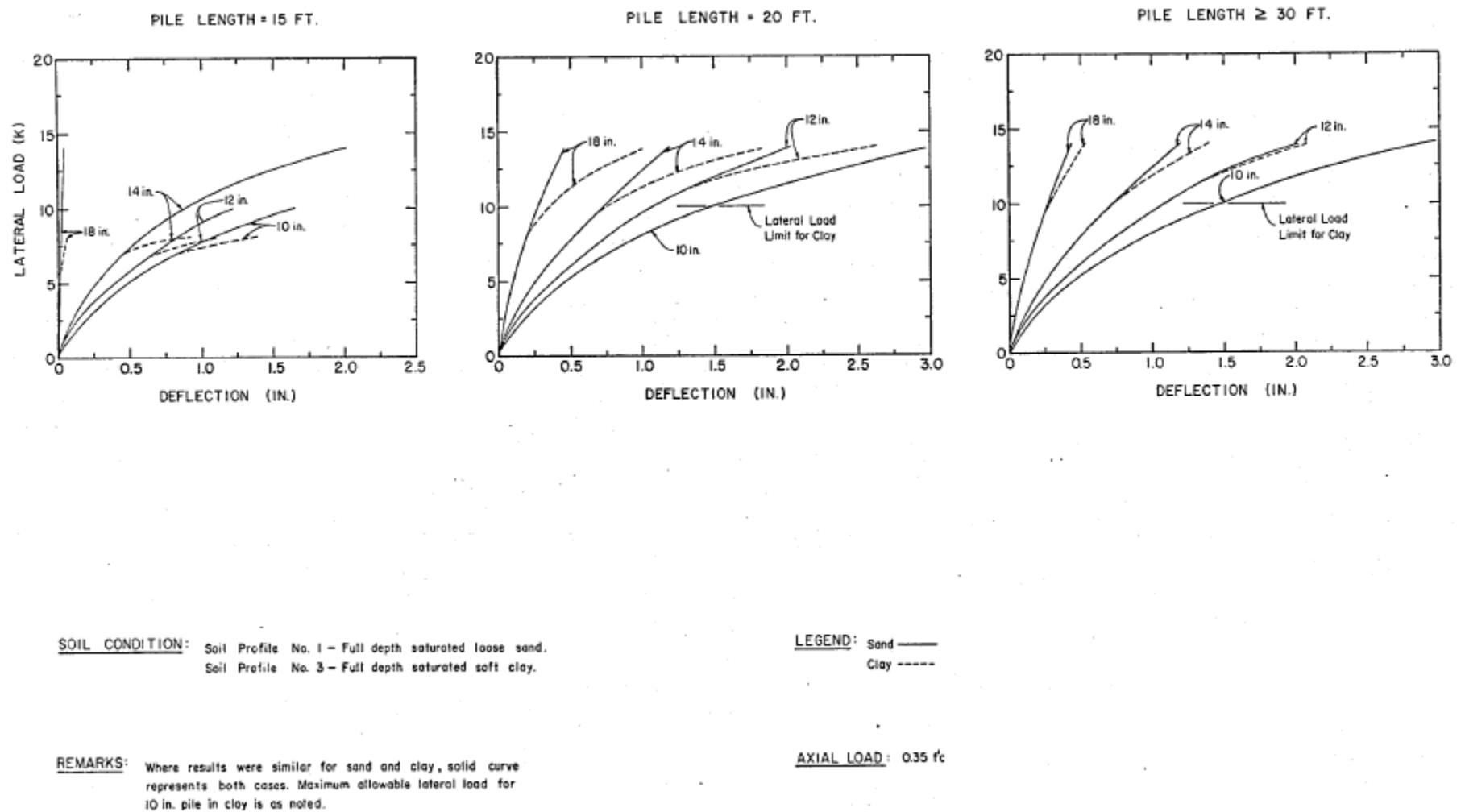


Figure 1.1.8-12 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 1 and 3

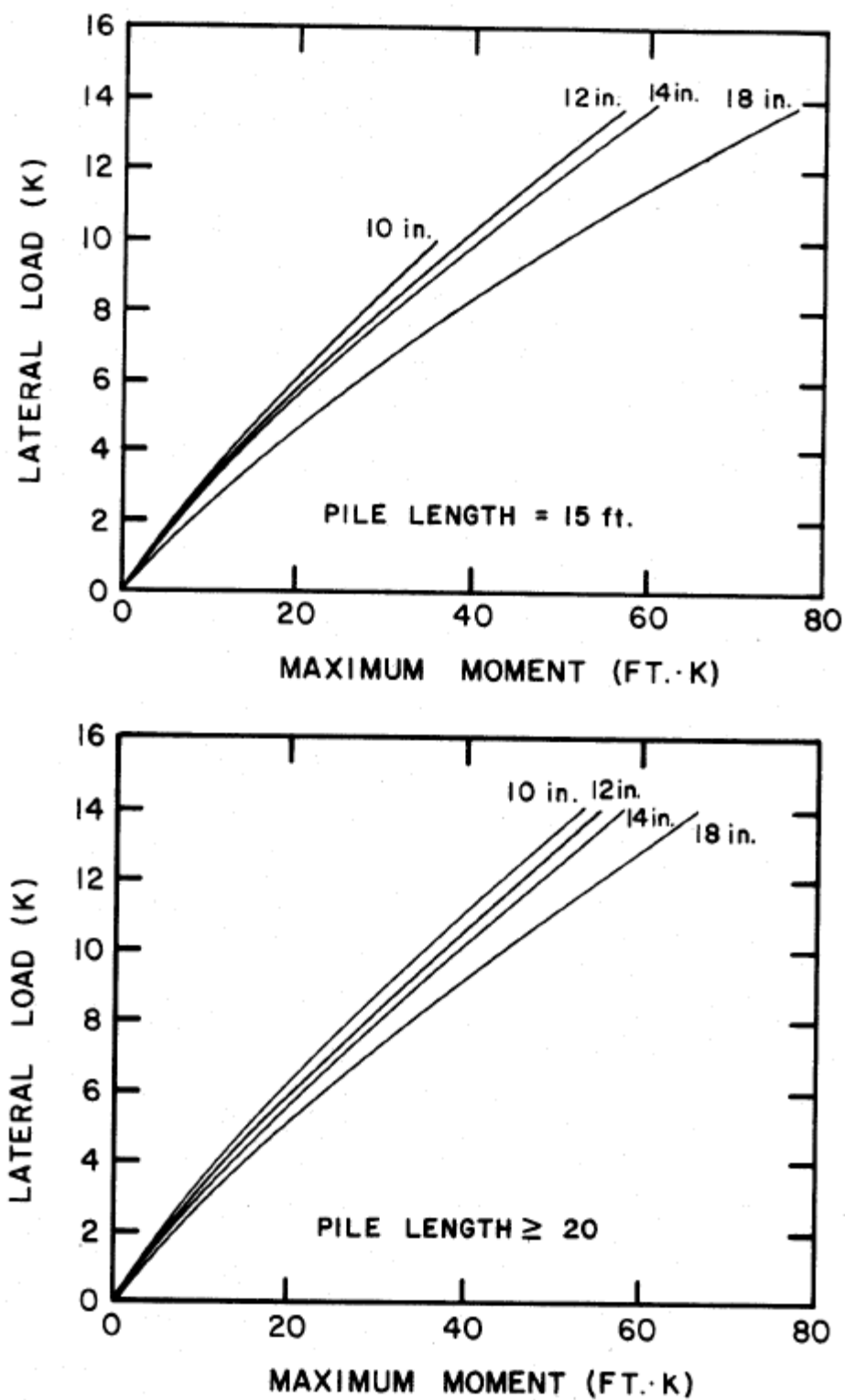


Figure 1.1.8-13 - Load Versus Maximum Moment for Cast-in-place Pile, Soil Profiles 1 and 3

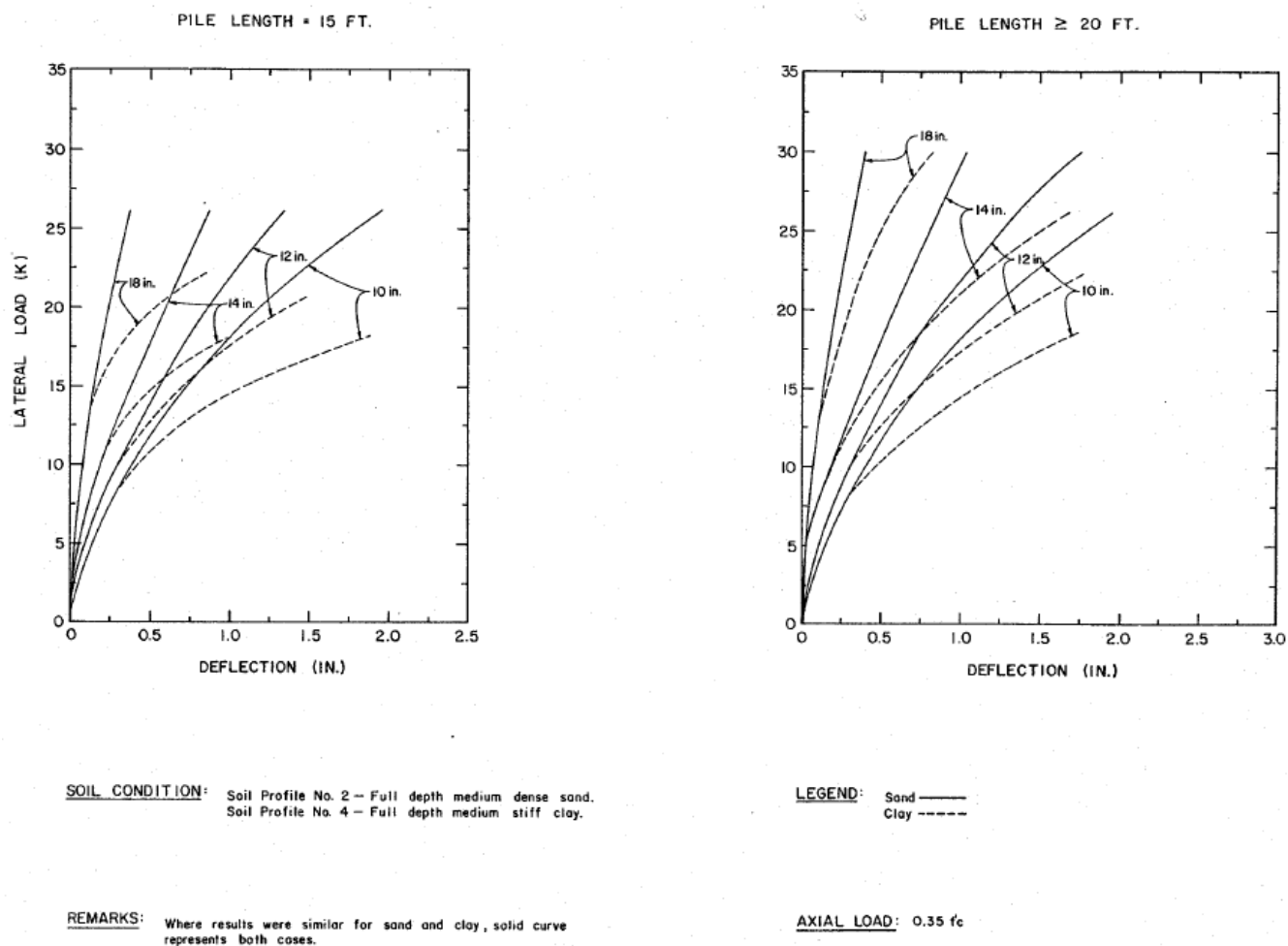


Figure 1.1.8-14 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 2 and 4

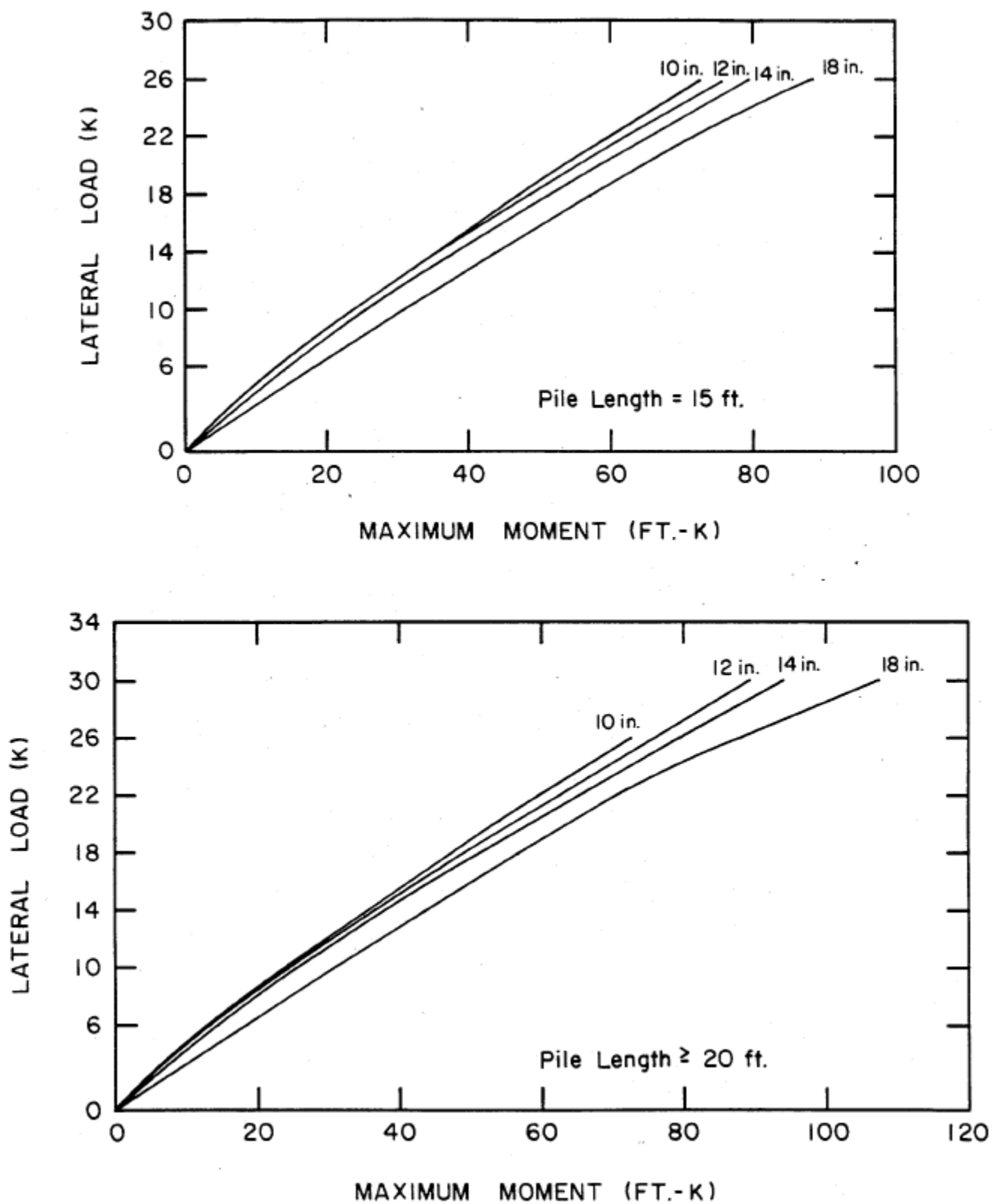


Figure 1.1.8-15 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 2

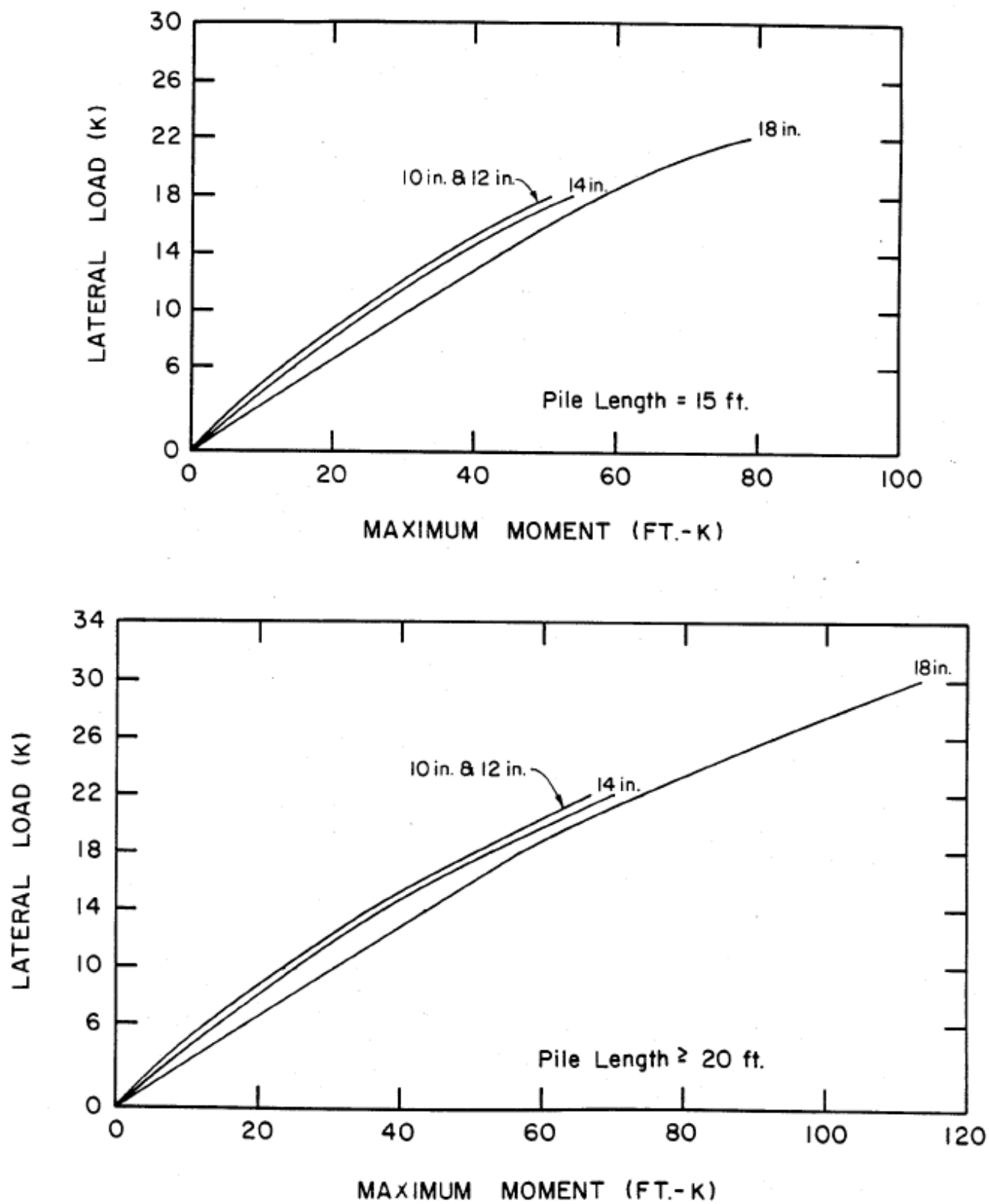


Figure 1.1.8-16 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 4

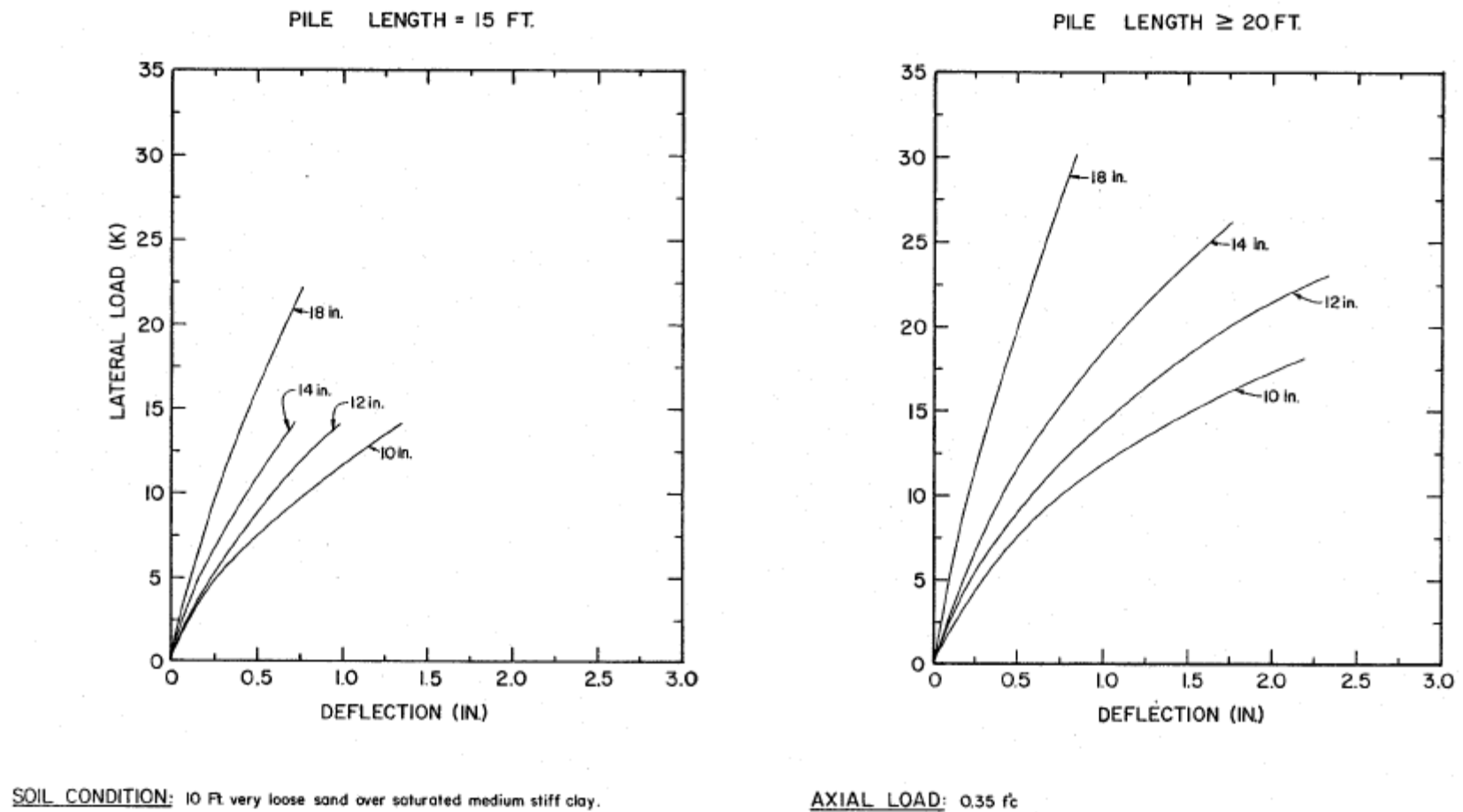


Figure 1.1.8-17 - Load Versus Deflection for Cast-in-Place Pile, Soil Profile 5

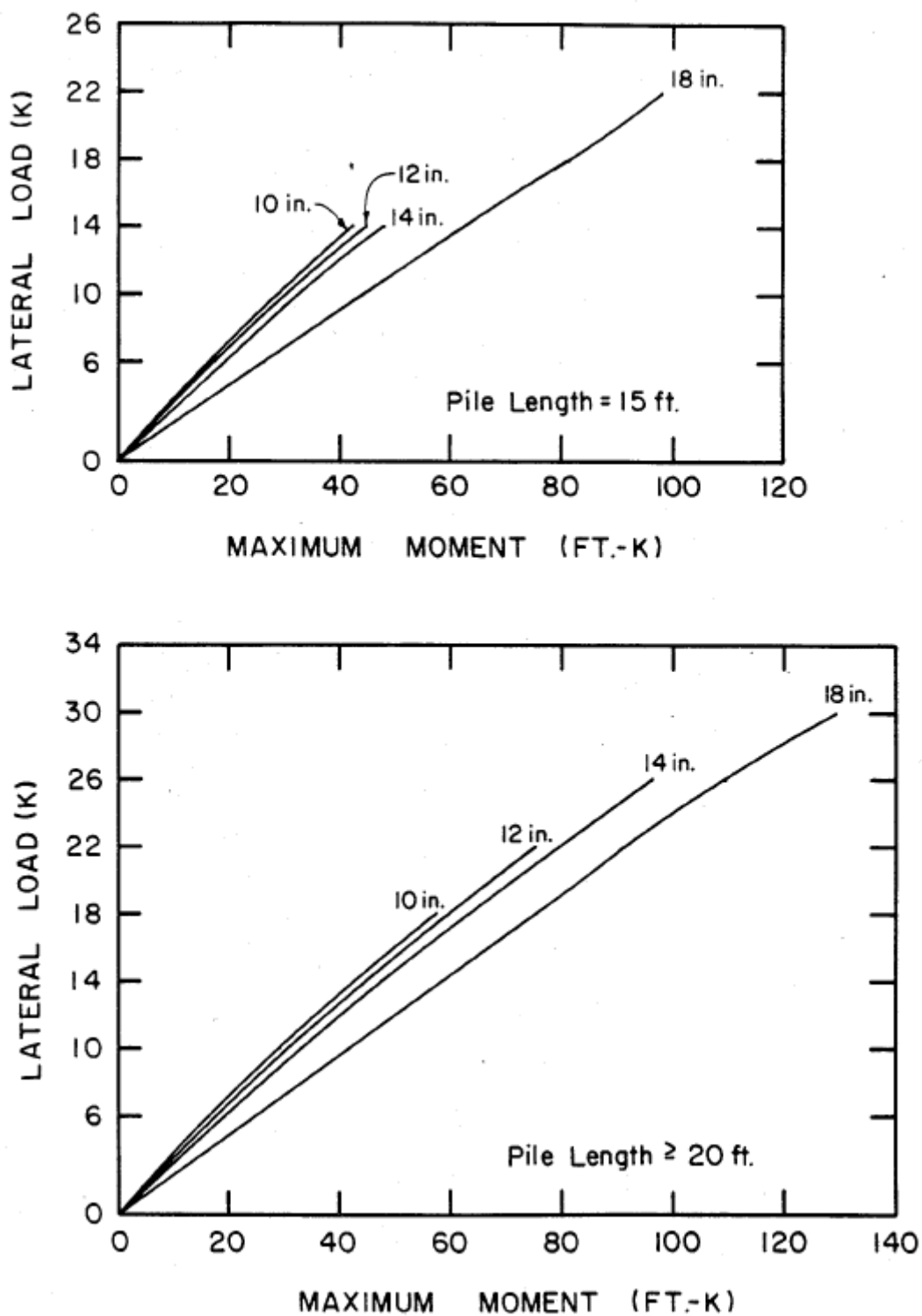


Figure 1.1.8-18 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 5

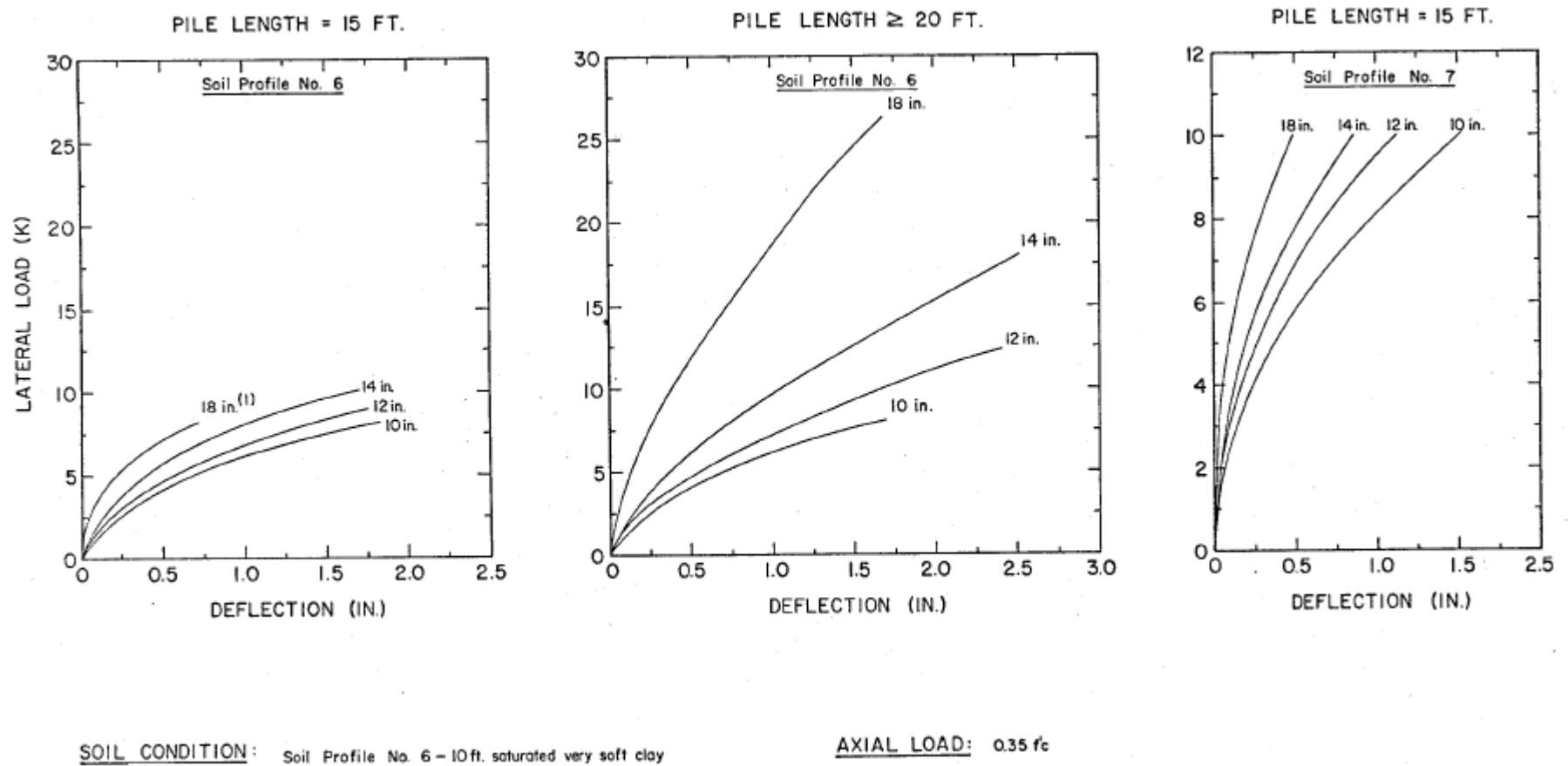


Figure 1.1.8-19 - Load Versus Deflection for Cast-in-Place Pile, Soil Profiles 6 and 7

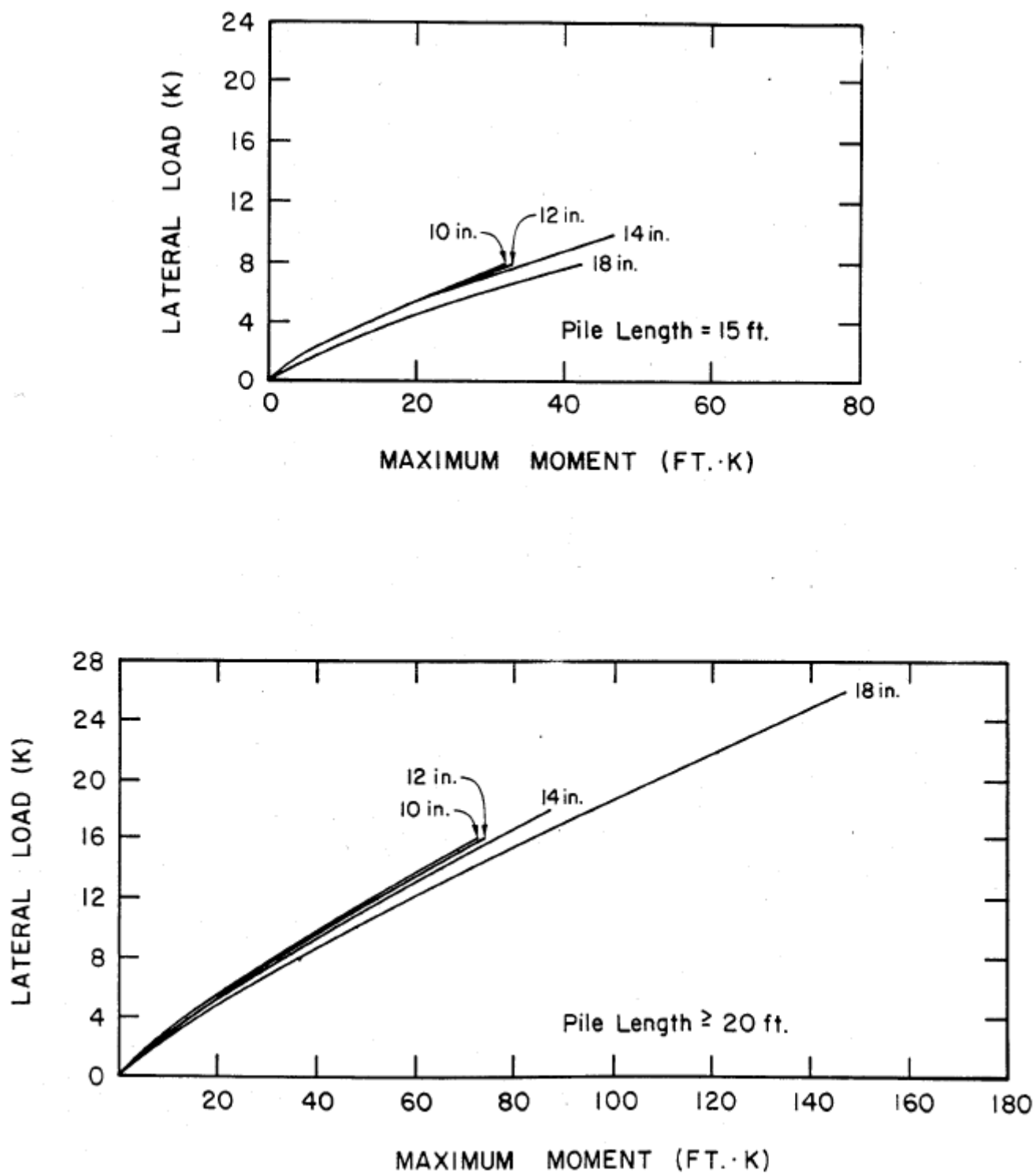


Figure 1.1.8-20 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 6

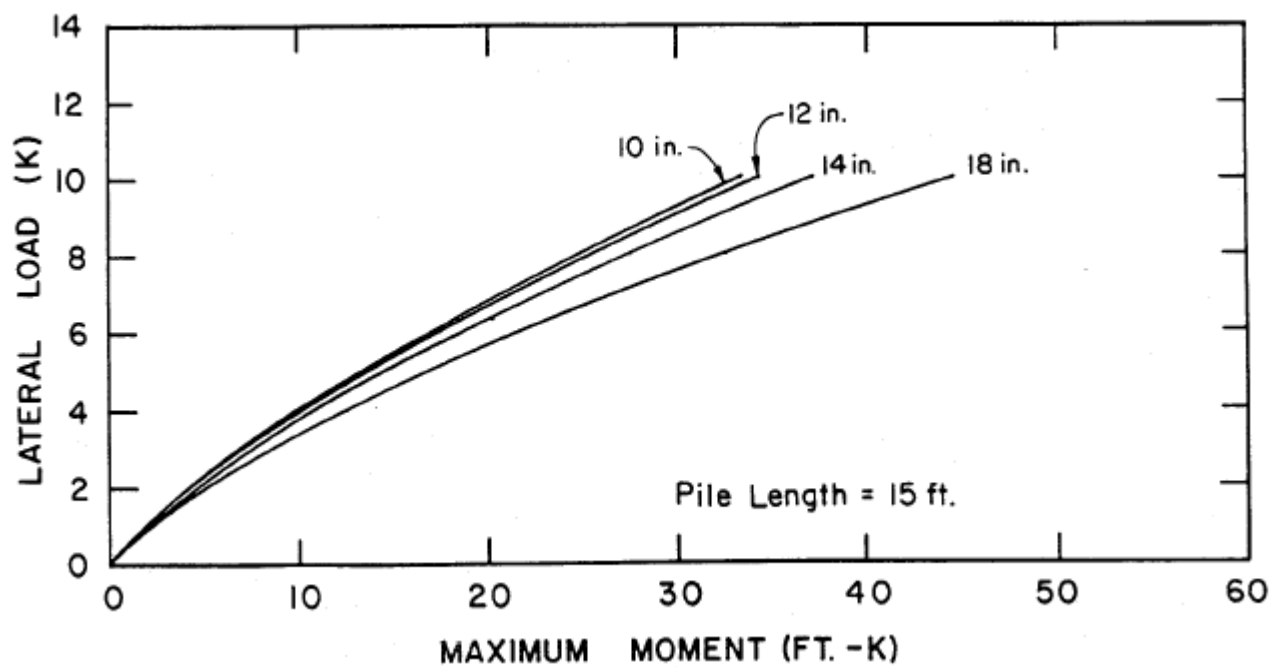


Figure 1.1.8-21 - Load Versus Maximum Moment for Cast-in-Place Pile, Soil Profile 7

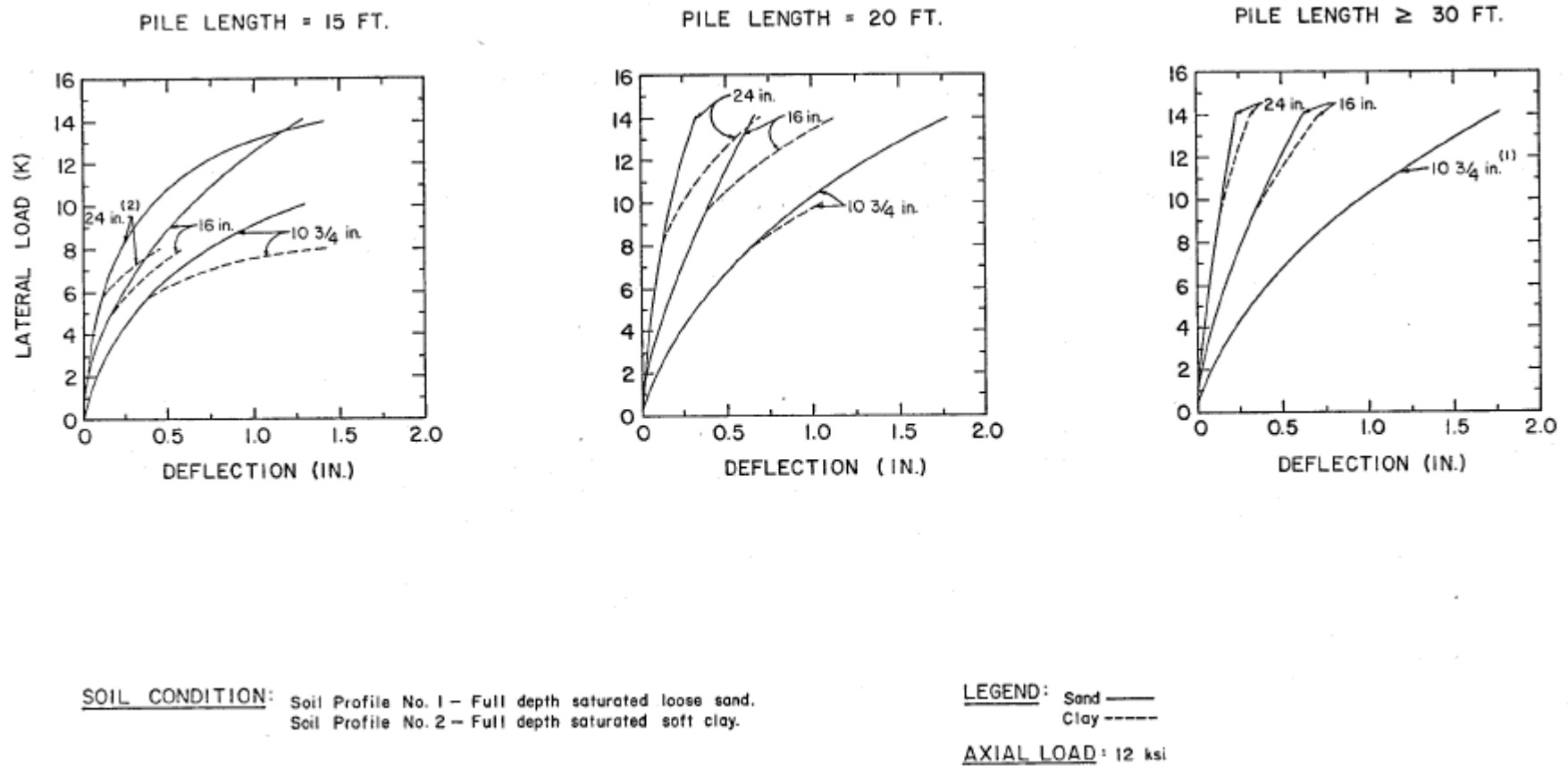


Figure 1.1.8-22 - Load Versus Deflection for Pipe Pile, Soil Profiles 1 and 3

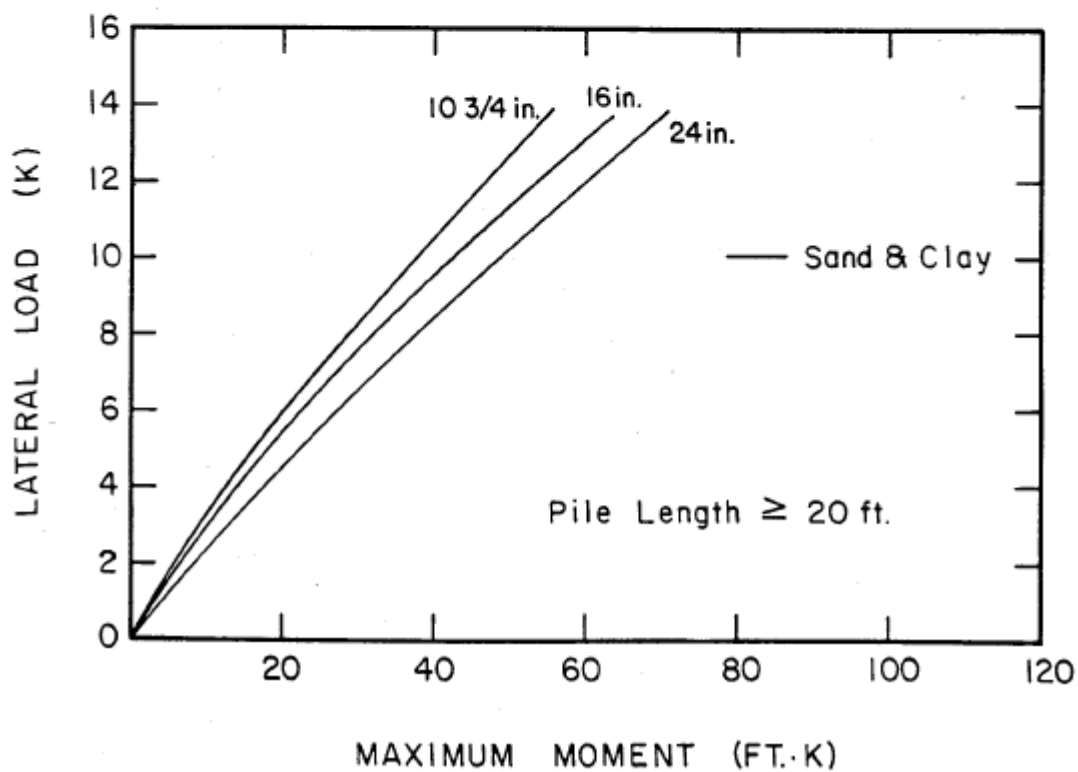
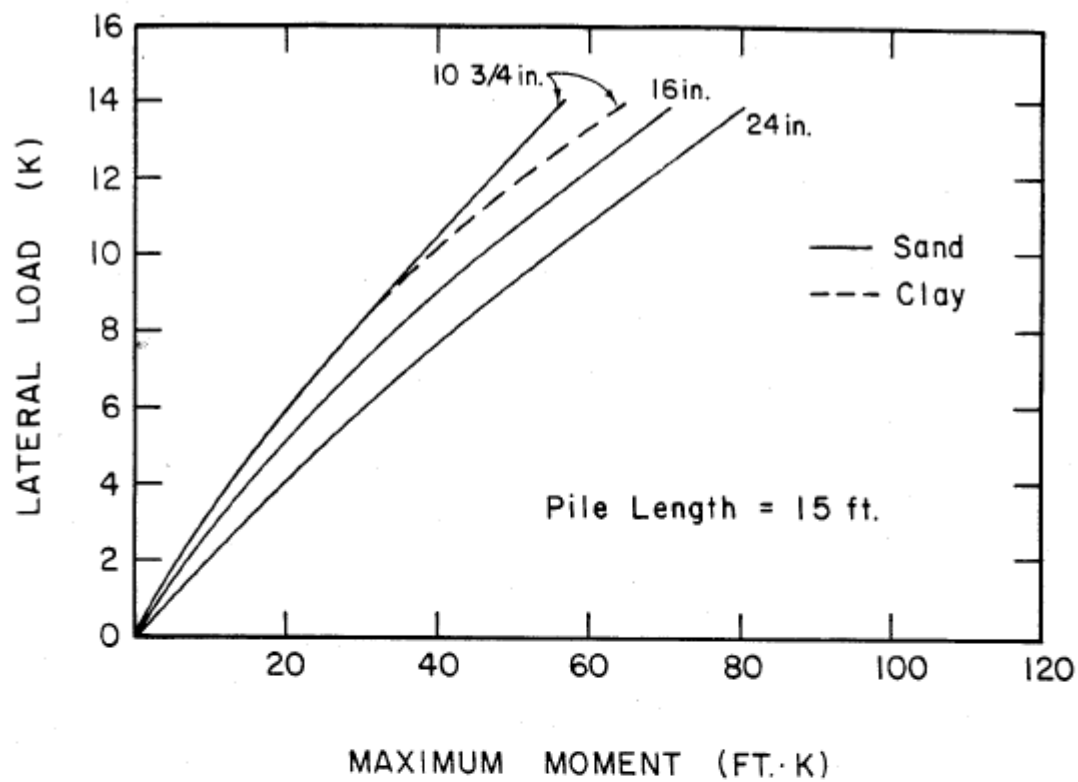
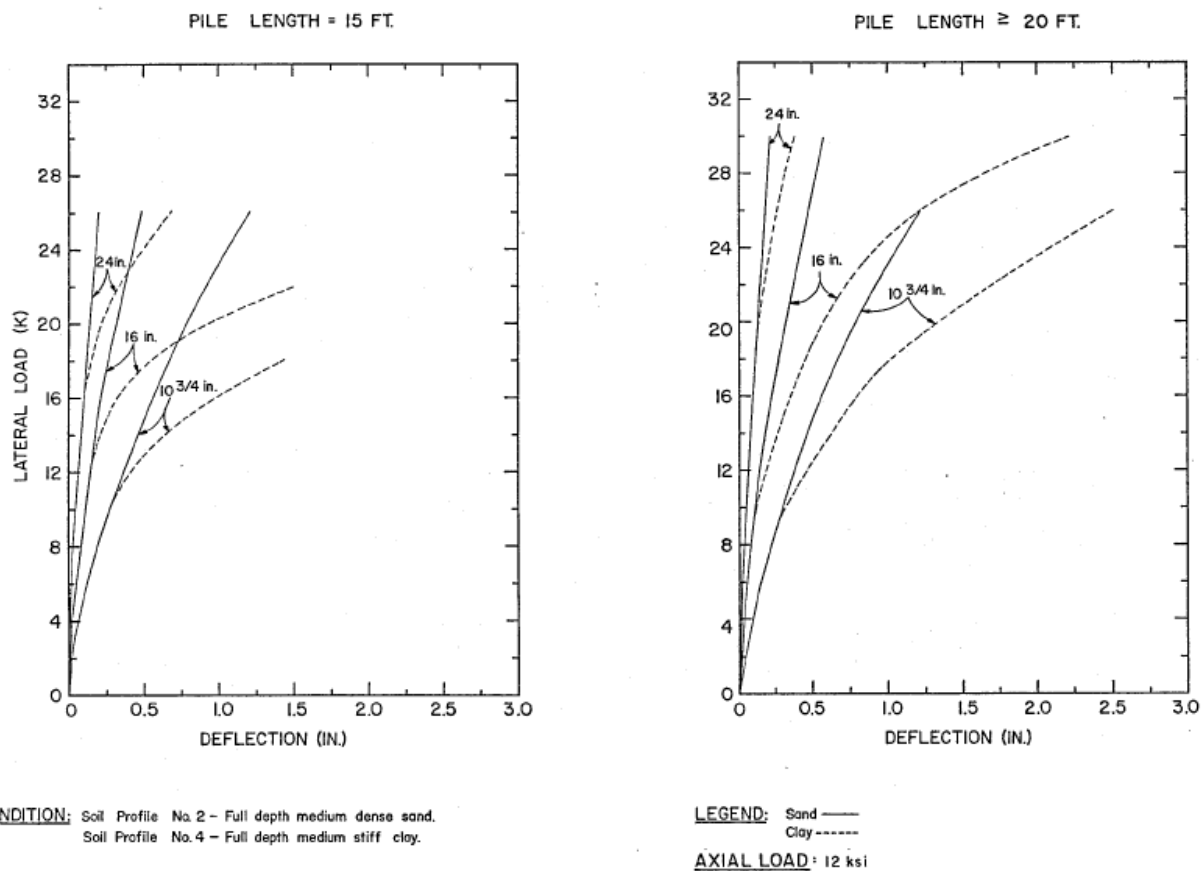


Figure 1.1.8-23 - Load Versus Maximum Moment for Pipe Pile, Soil Profiles 1 and 3



REMARKS: Where results were similar for sand and clay, solid curve represents both cases.

Figure 1.1.8-24 - Load Versus Deflection for Pipe Pile, Soil Profiles 2 and 4

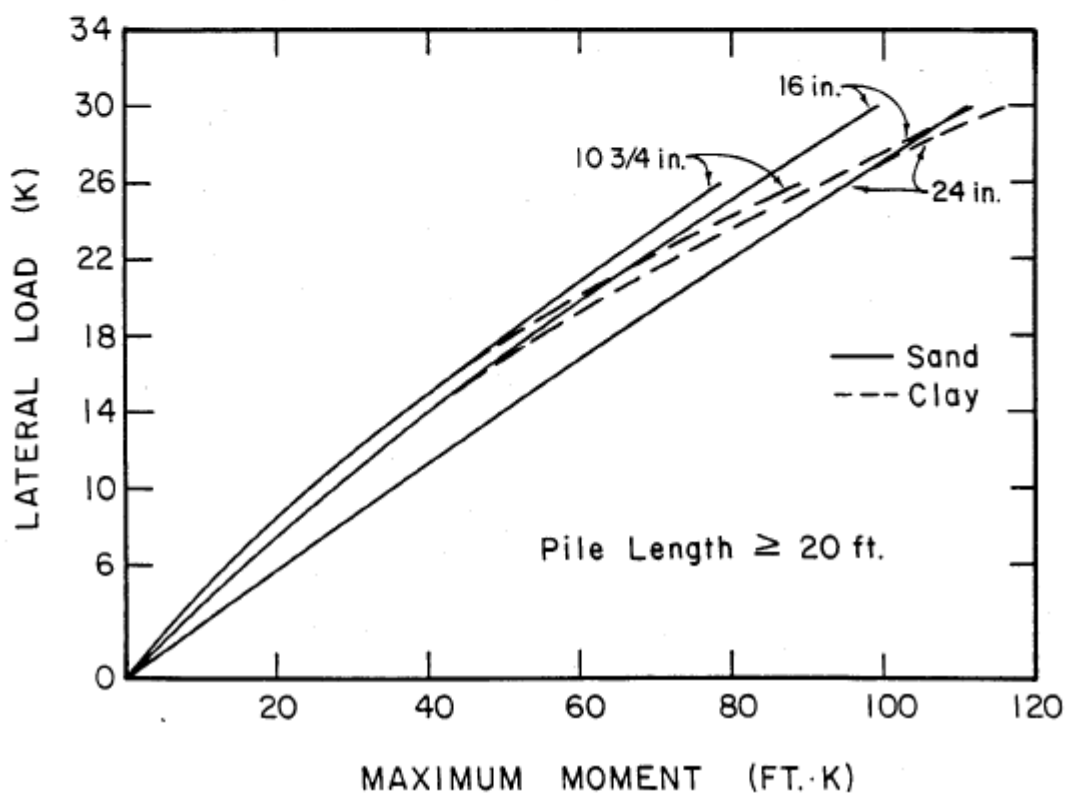
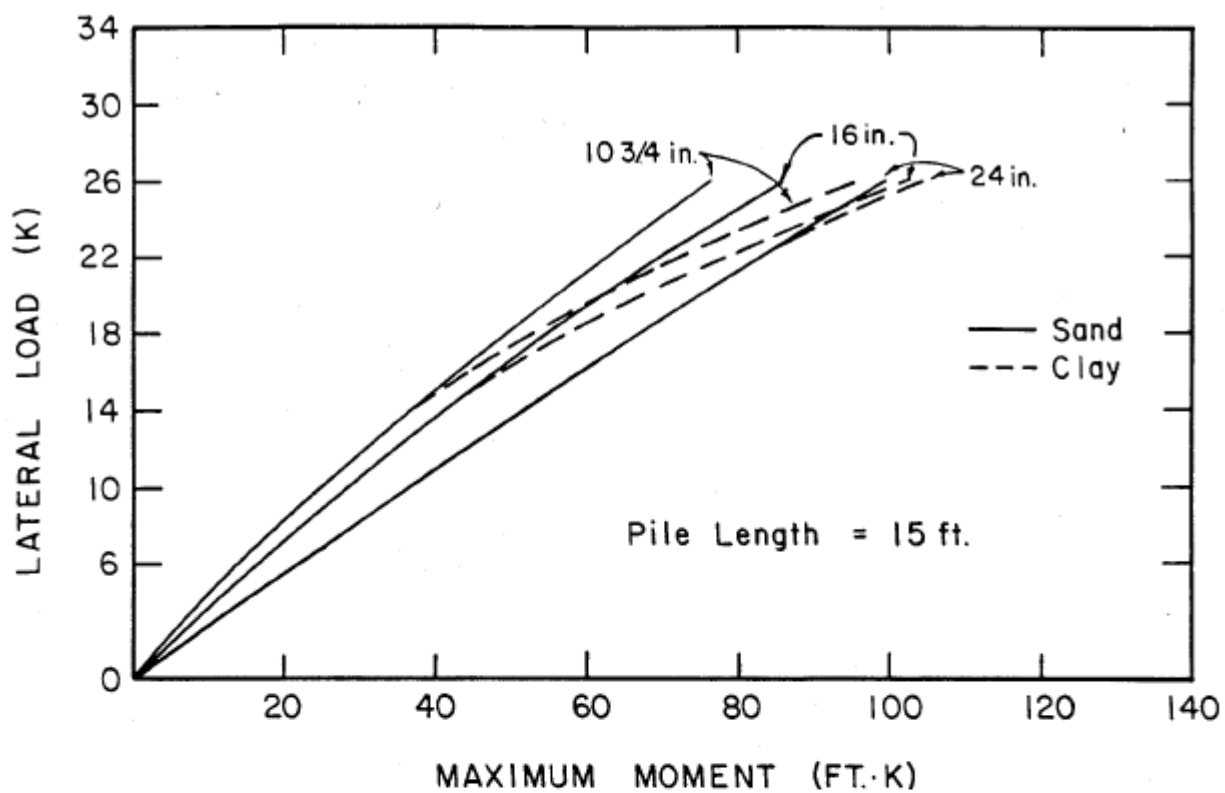


Figure 1.1.8-25 - Load Versus Maximum Moment for Pipe Pile, Soil Profiles 2 and 4

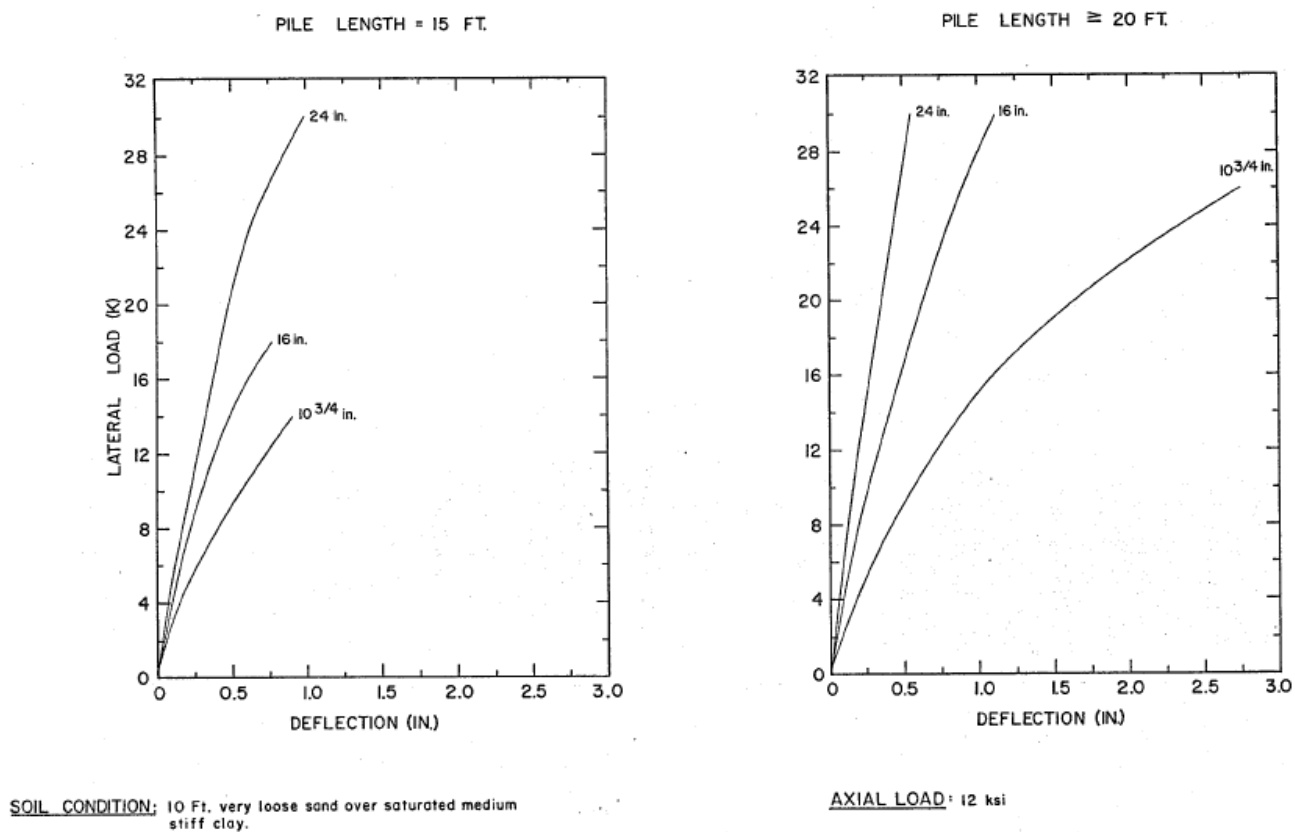


Figure 1.1.8-26 - Load Versus Deflection for Pipe Pile, Soil Profile 5

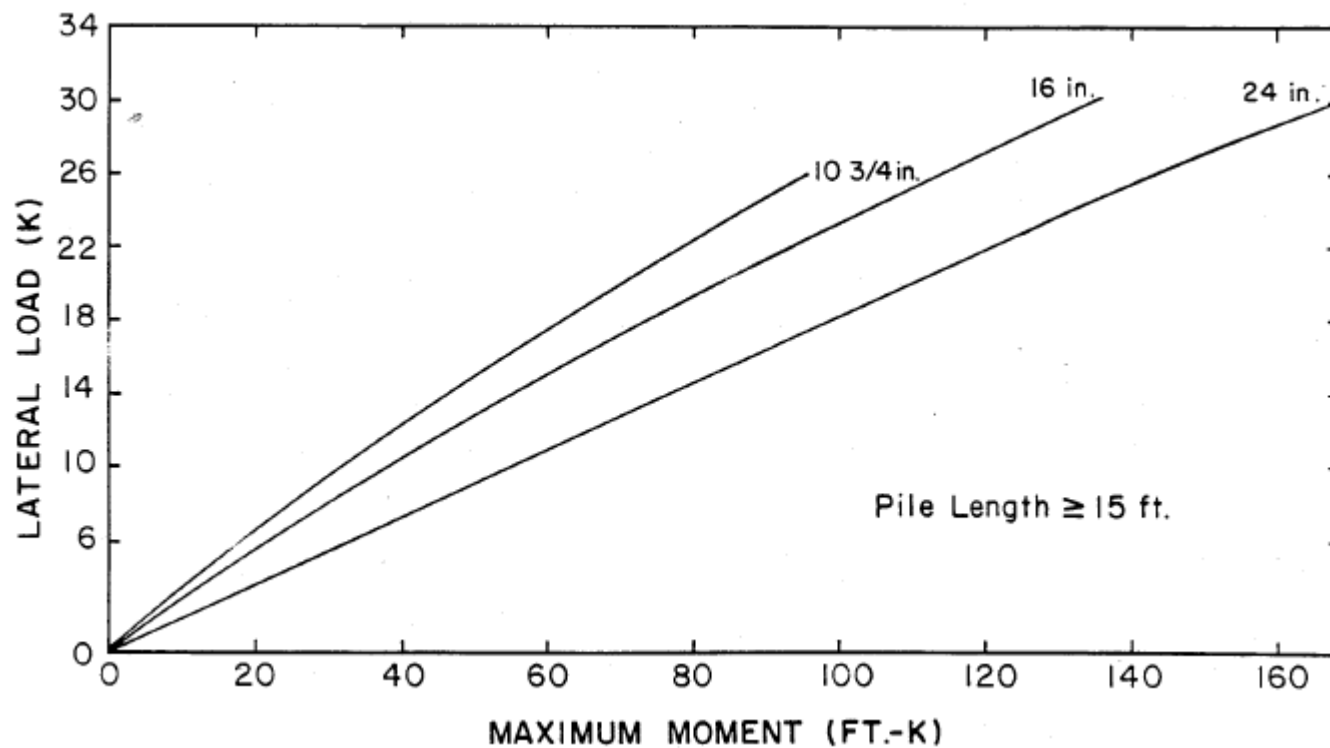


Figure 1.1.8-27 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 5

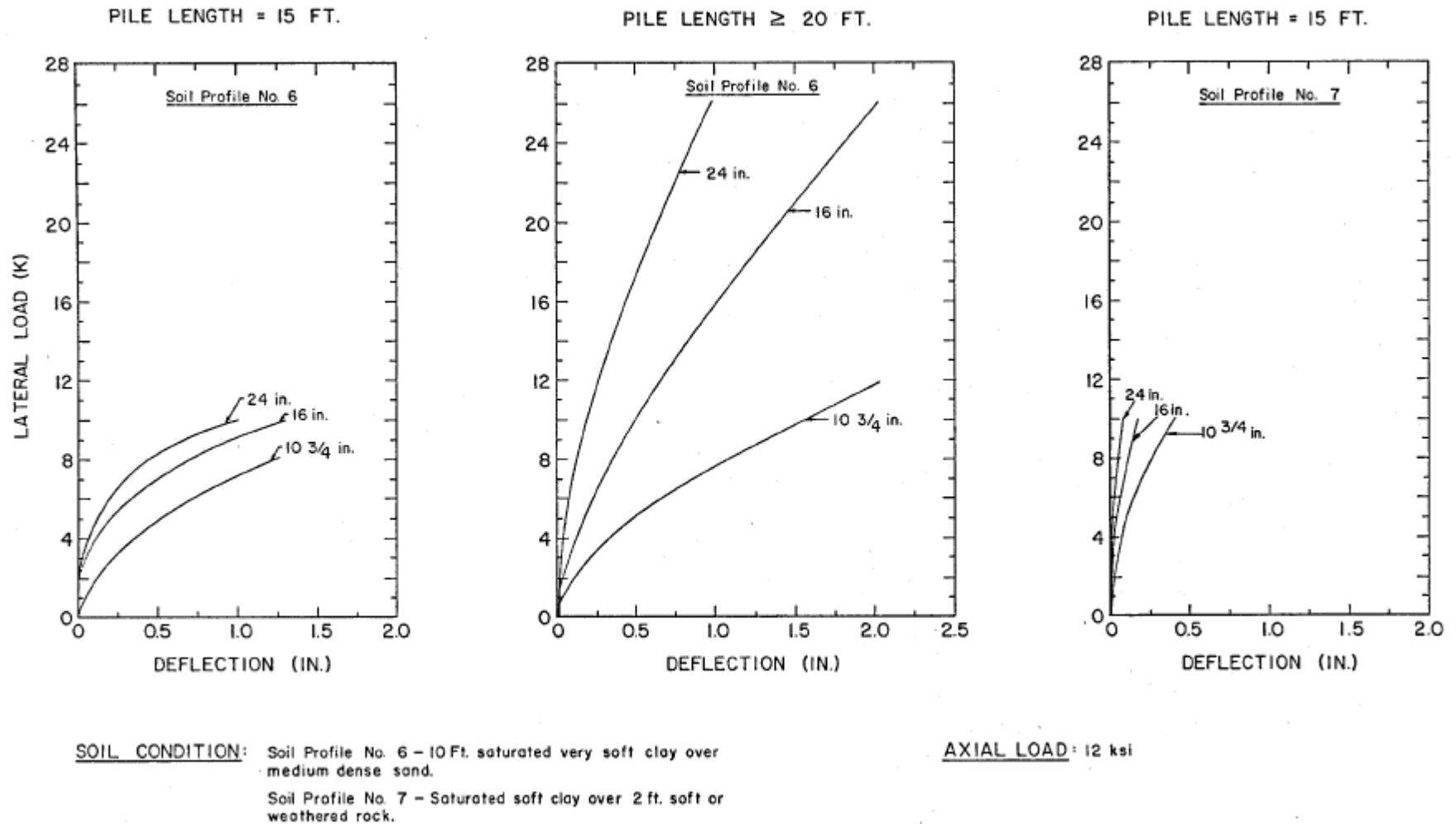


Figure 1.1.8-28 - Load Versus Deflection for Pipe Pile, Soil Profiles 6 and 7

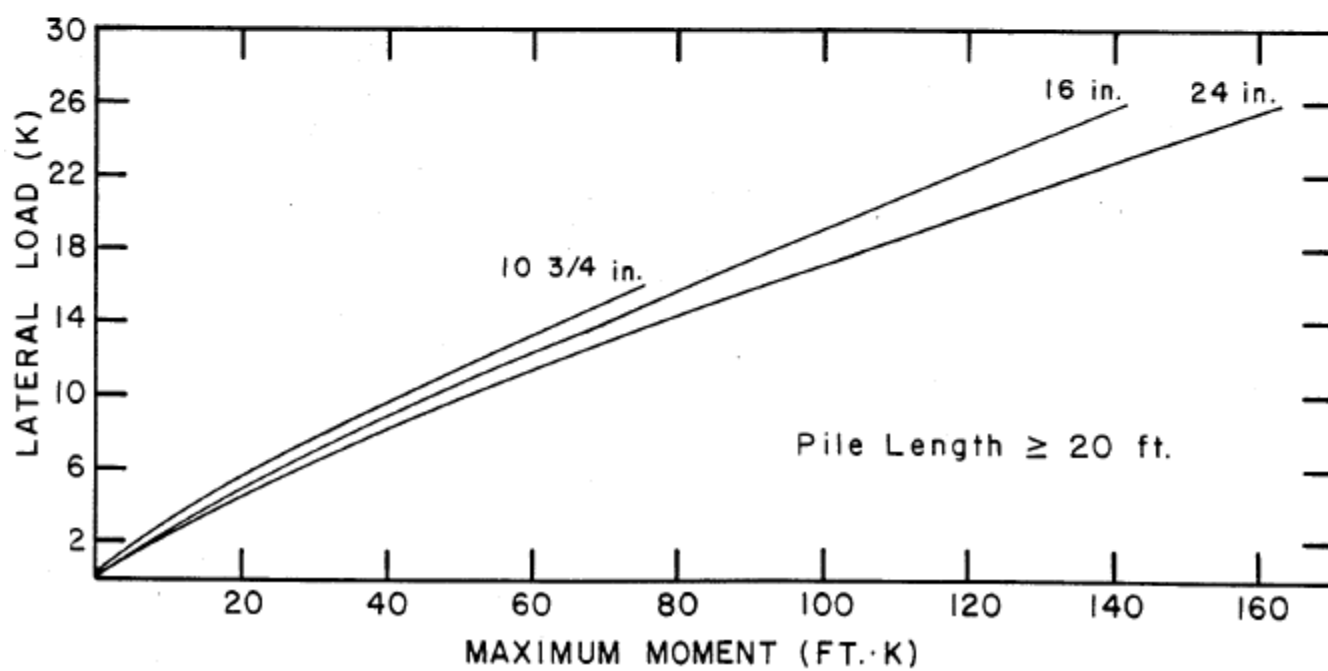
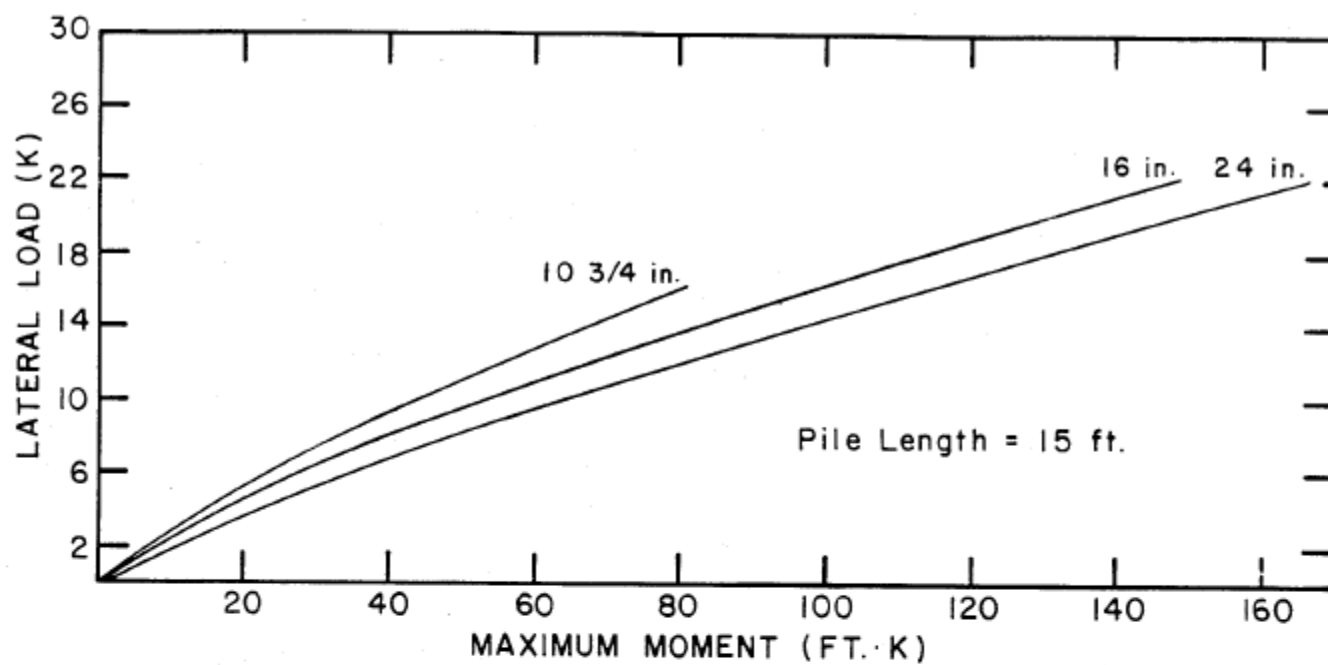


Figure 1.1.8-29 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 6

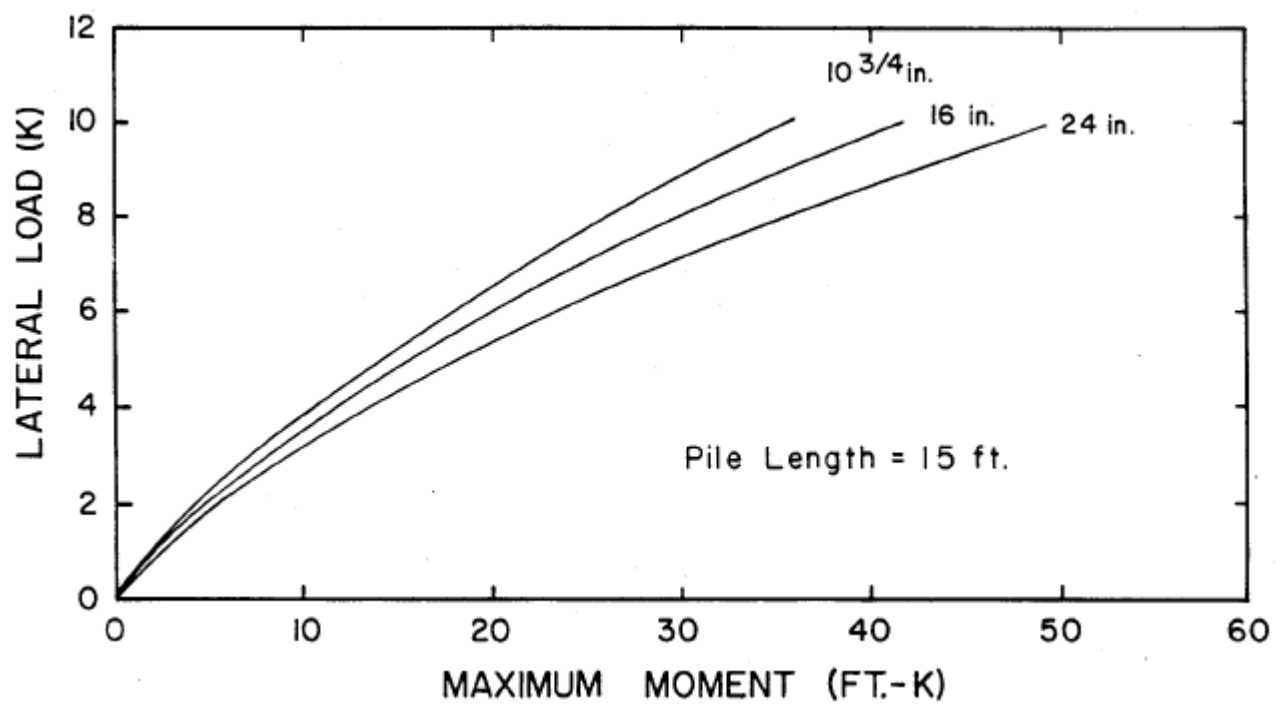


Figure 1.1.8-30 - Load Versus Maximum Moment for Pipe Pile, Soil Profile 7

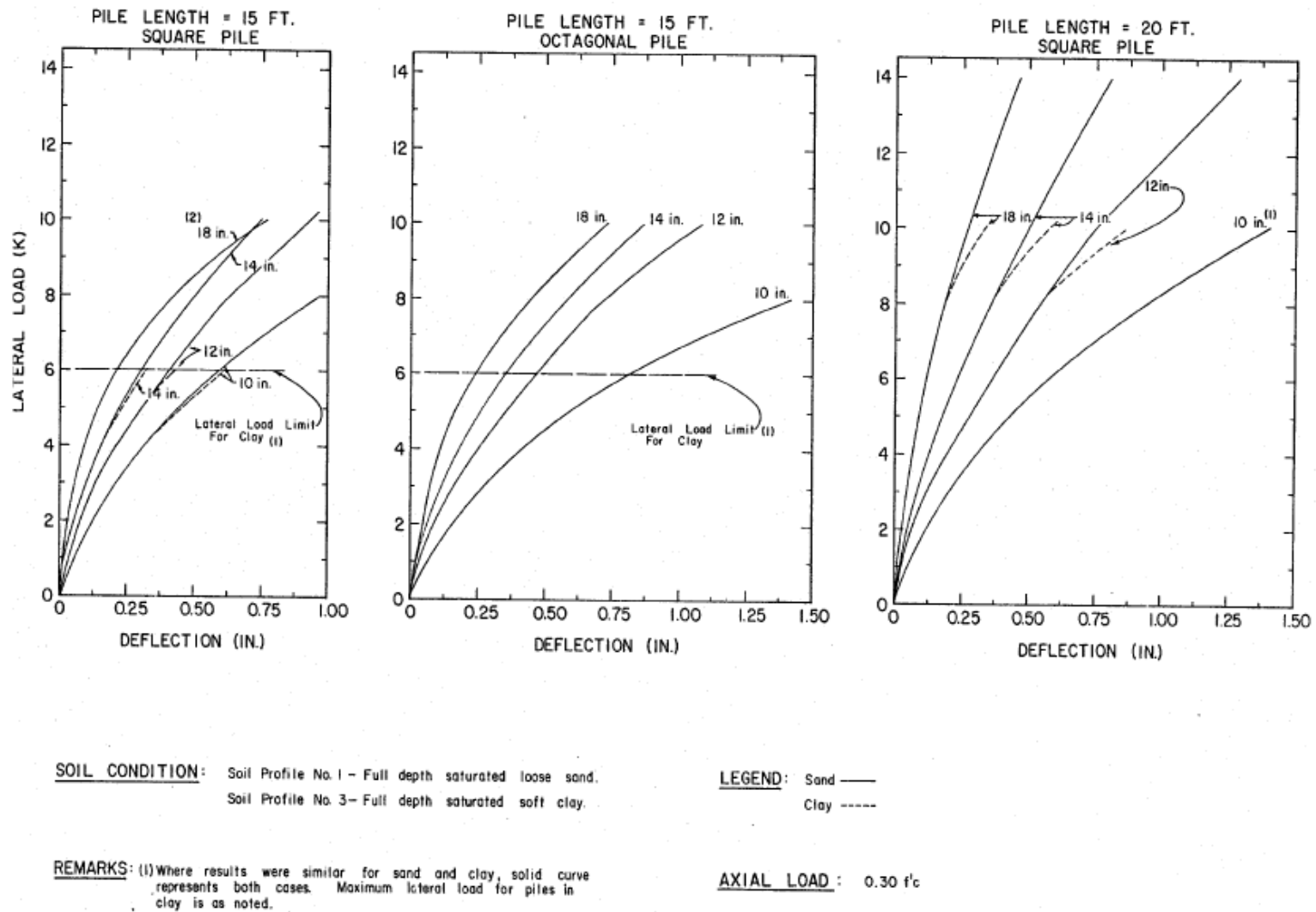


Figure 1.1.8-31 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 1 and 3

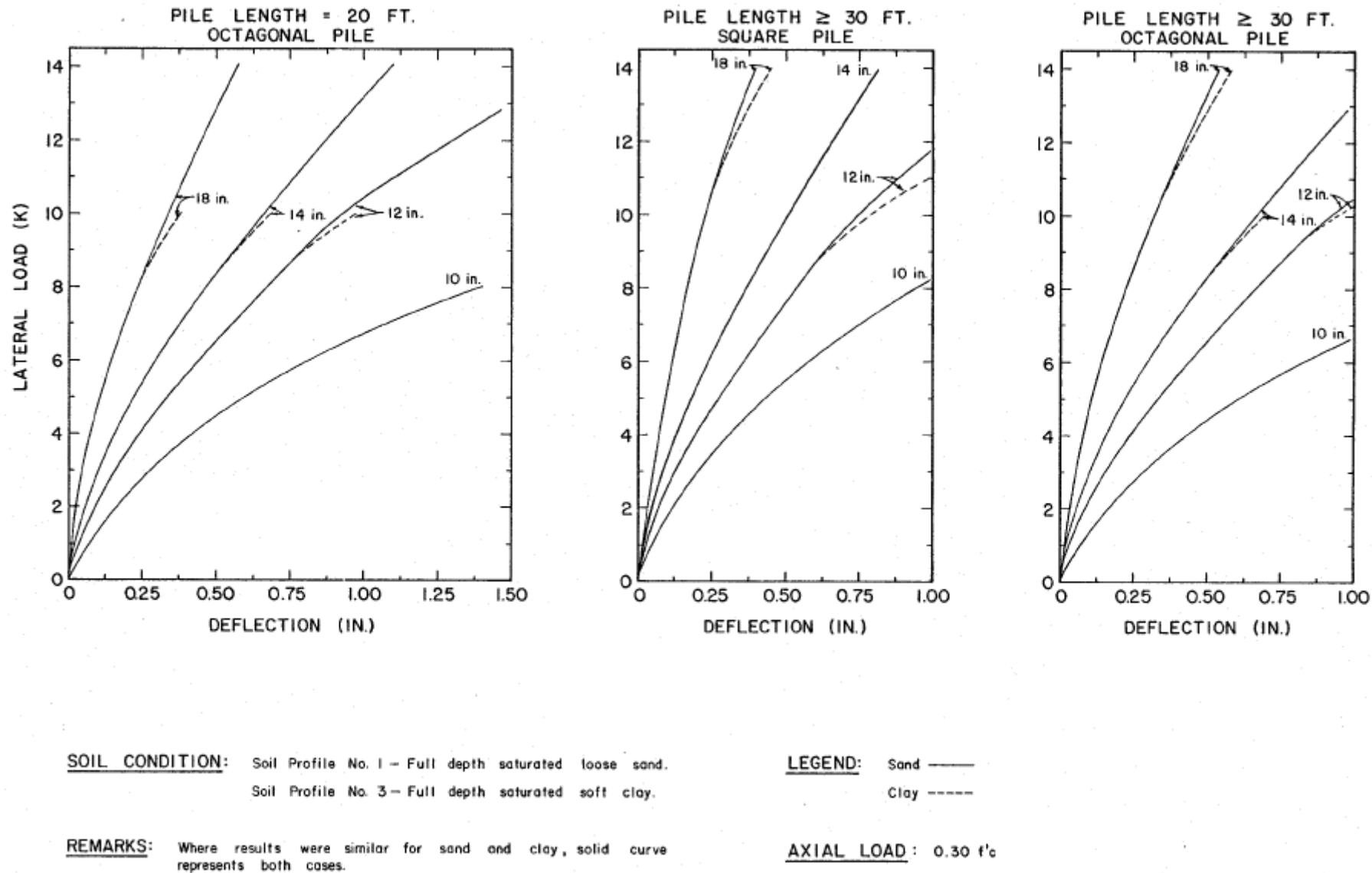
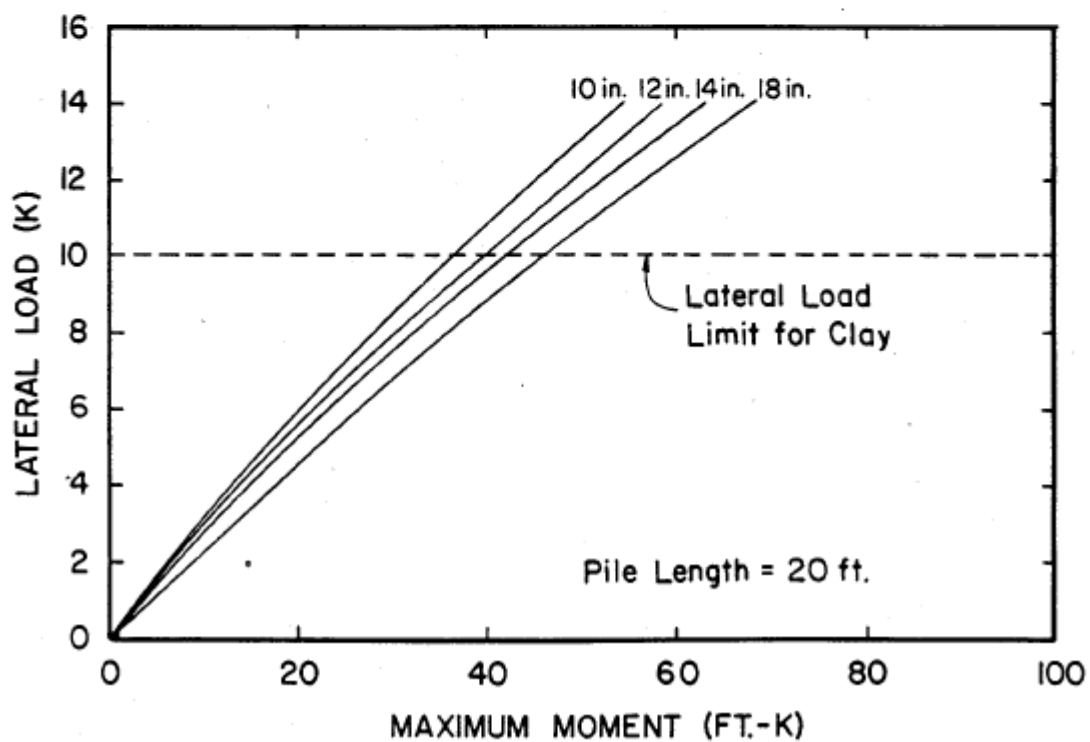
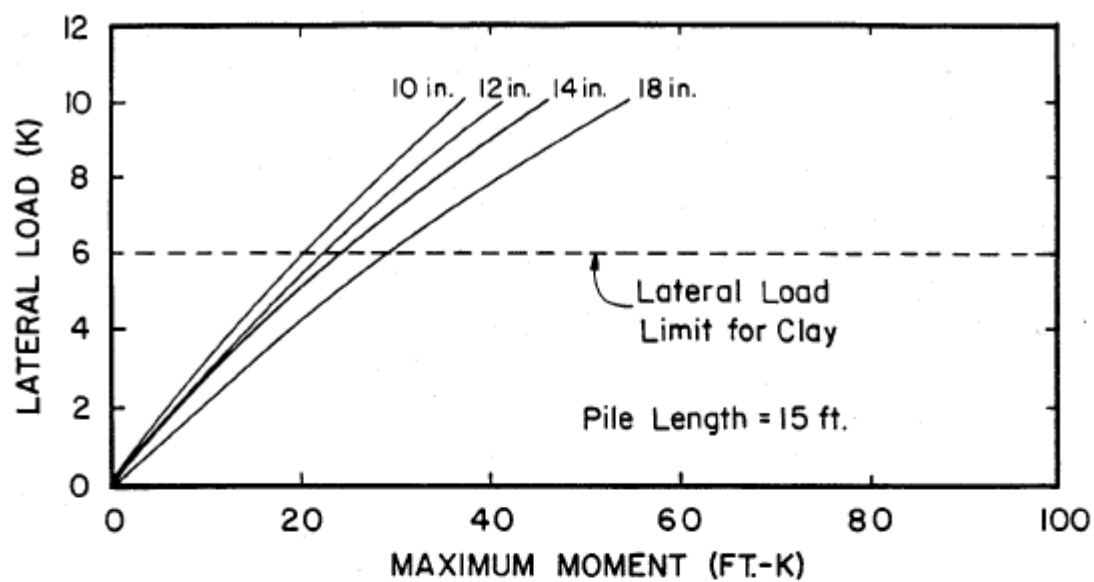
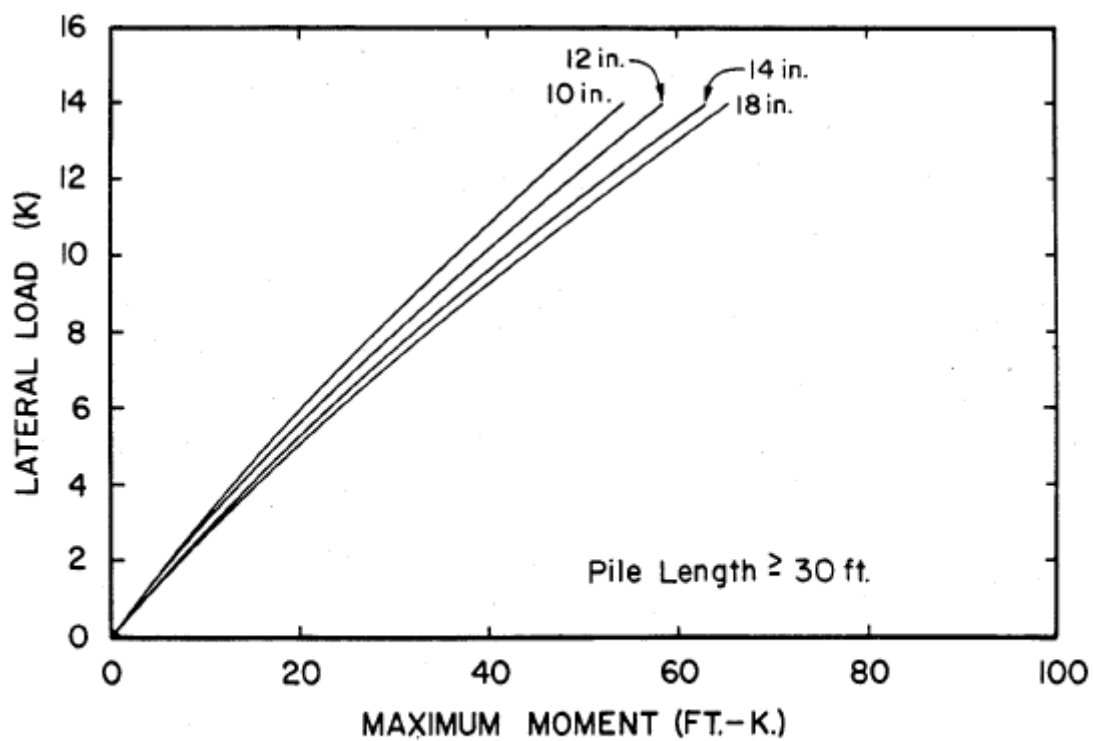


Figure 1.1.8-32 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 1 and 3



NOTE: Curves apply to both square and octagonal piles.

Figure 1.1.8-33 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 1 and 3



NOTE : Curves apply to both square and octagonal piles.

Figure 1.1.8-34 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 1 and 3

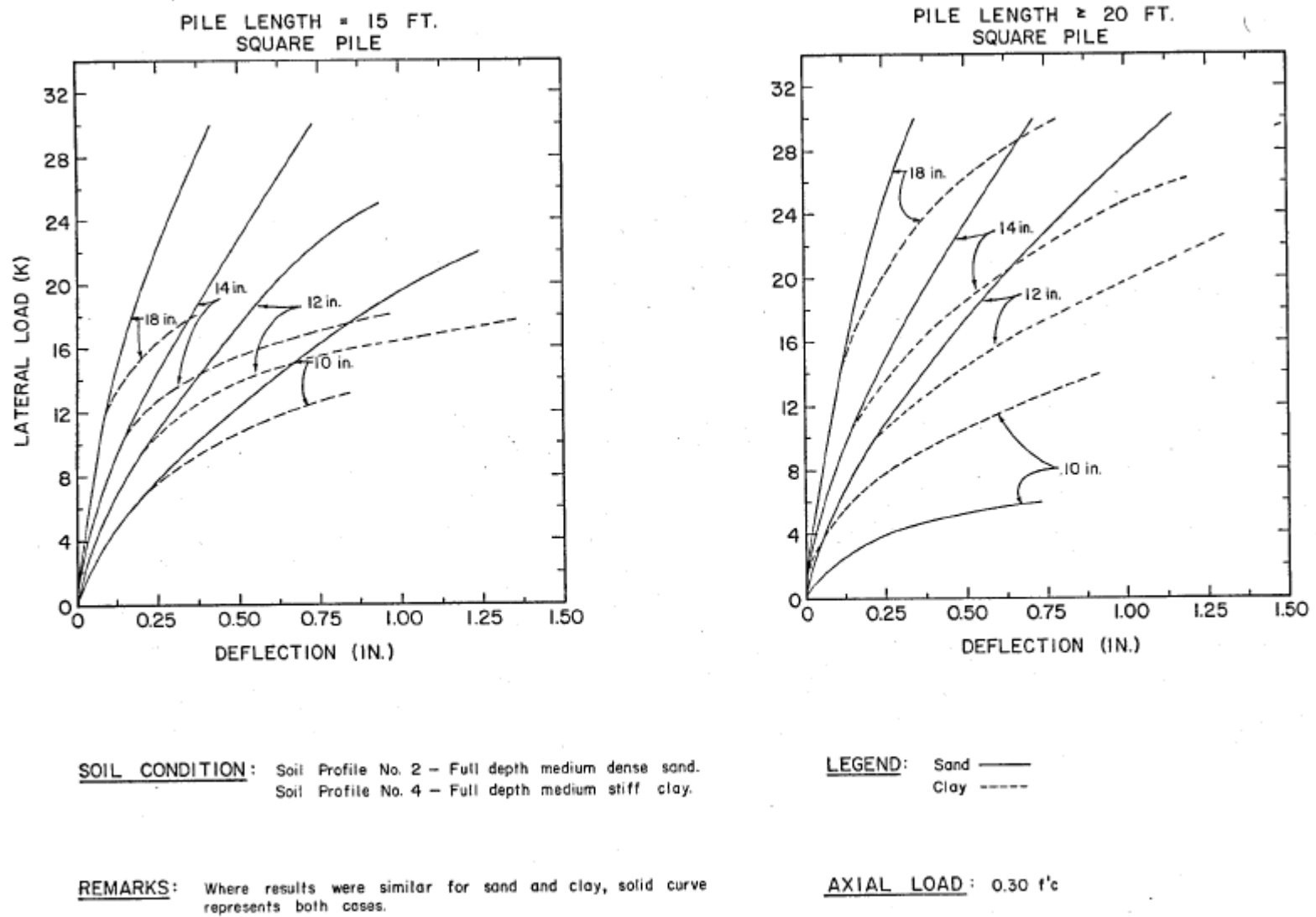


Figure 1.1.8-35 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 2 and 4

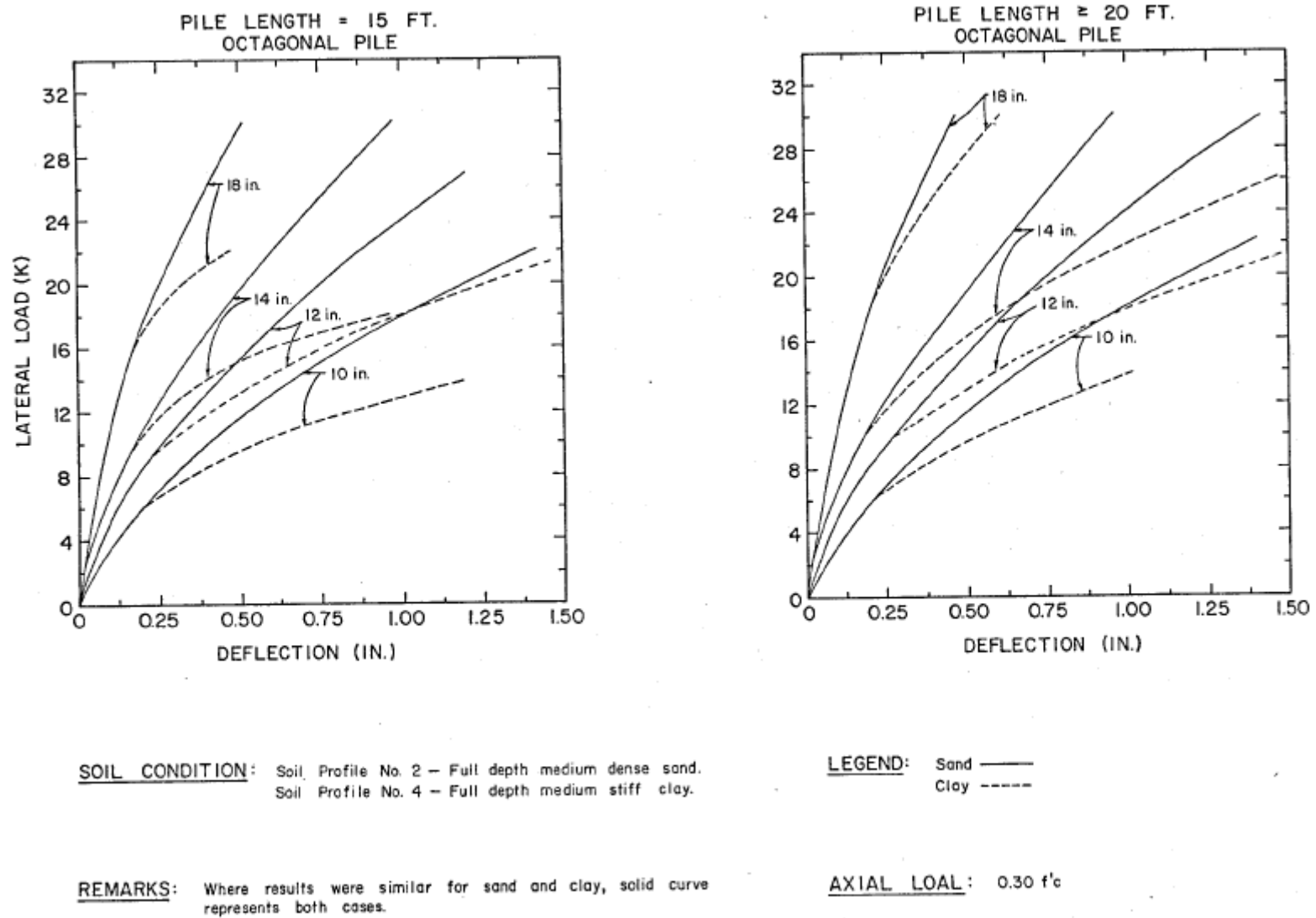


Figure 1.1.8-36 - Load Versus Deflection for Precast Concrete Pile, Soil Profiles 2 and 4

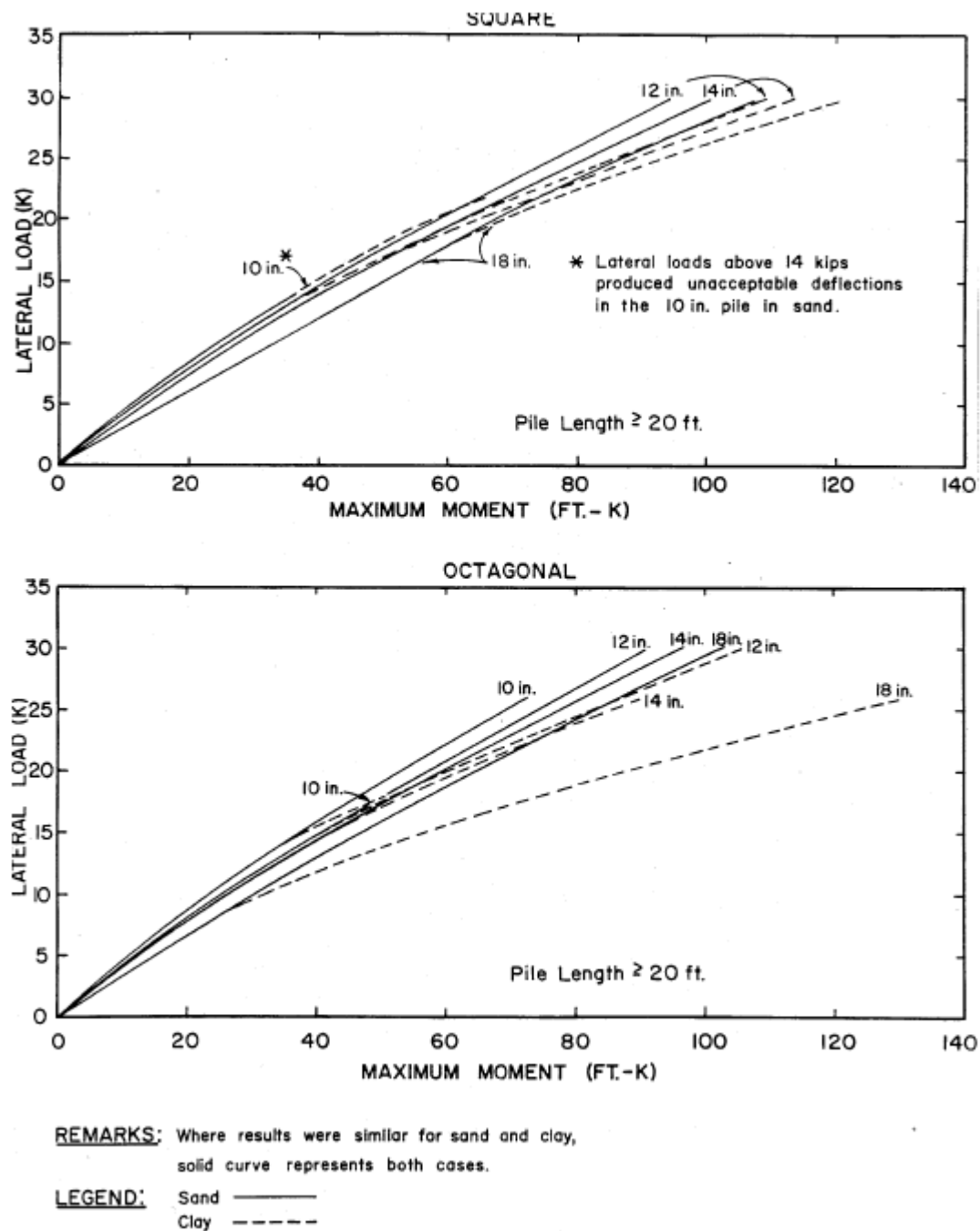
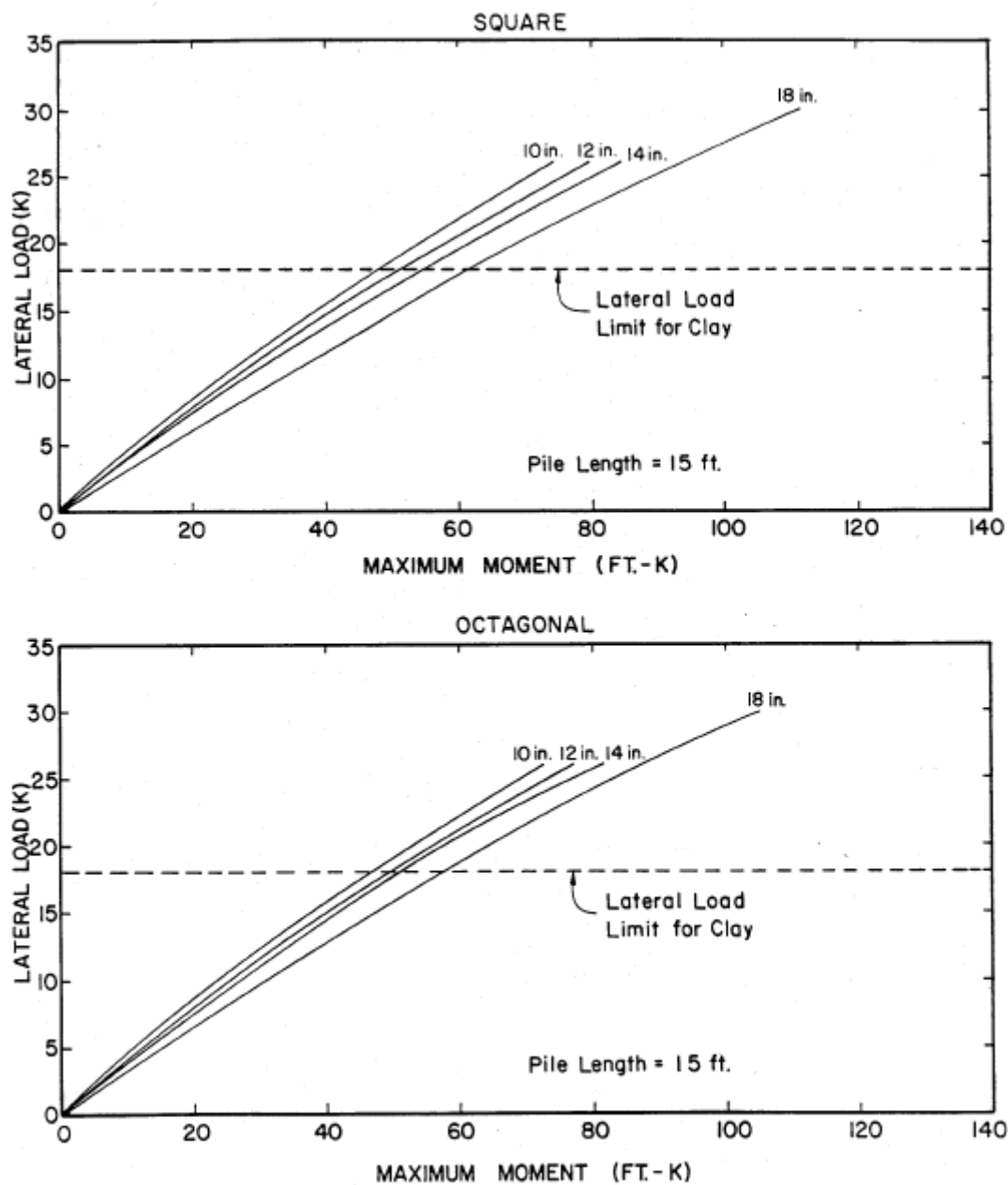


Figure 1.1.8-37 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 2 and 4



REMARKS: Where results were similar for sand and clay,
solid curve represents both cases.

Figure 1.1.8-38 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profiles 2 and 4

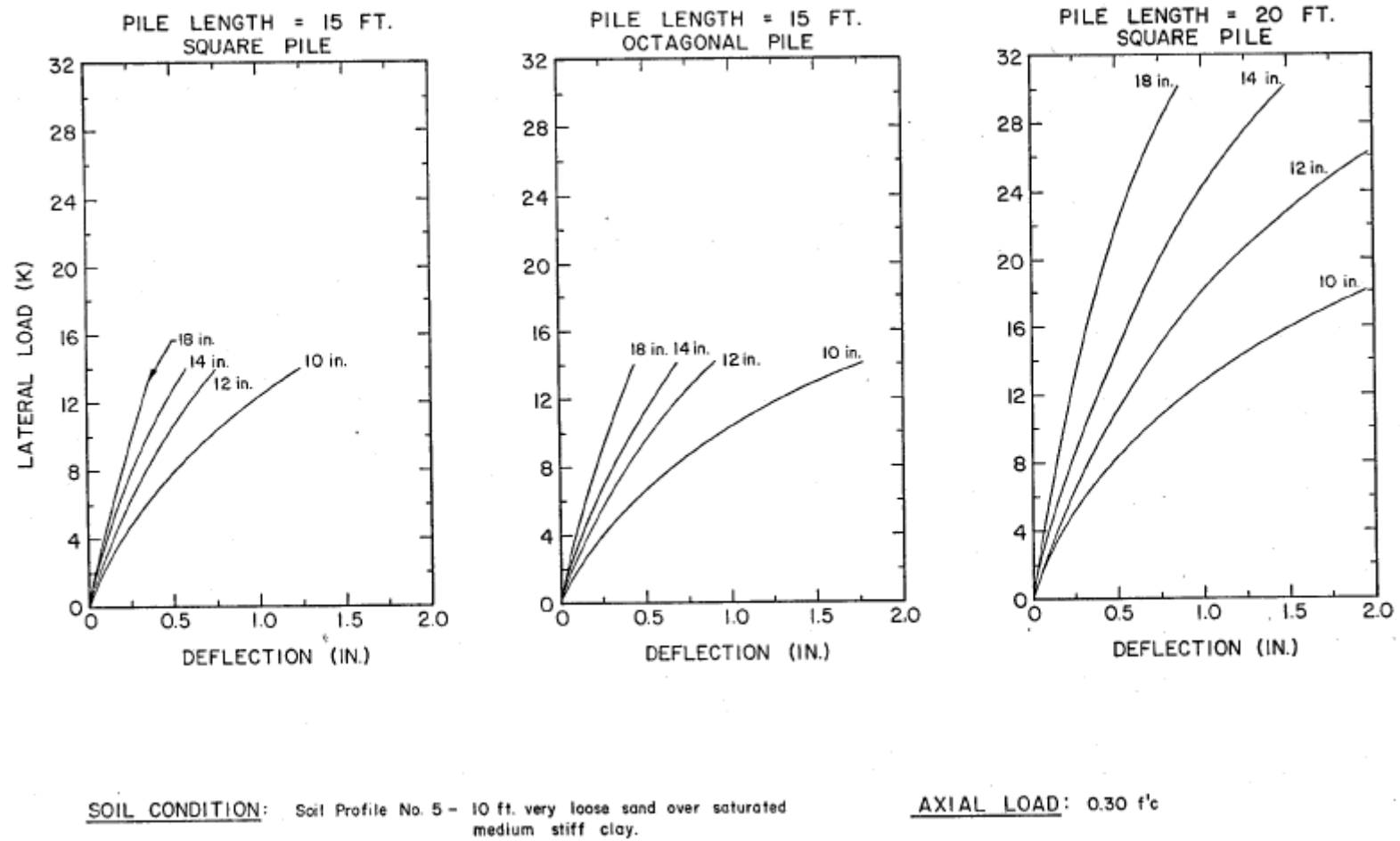


Figure 1.1.8-39 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 5

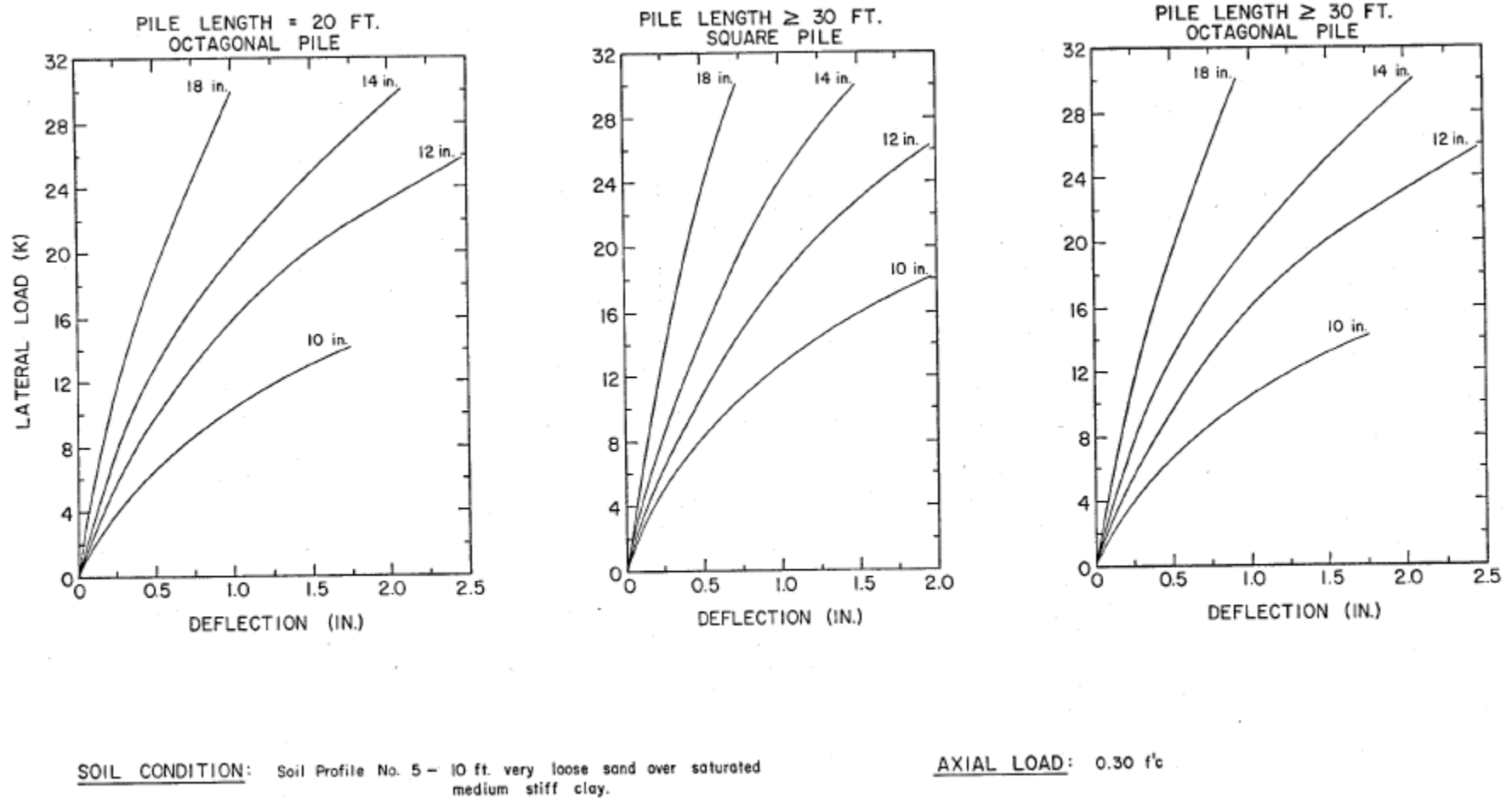


Figure 1.1.8-40 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 5

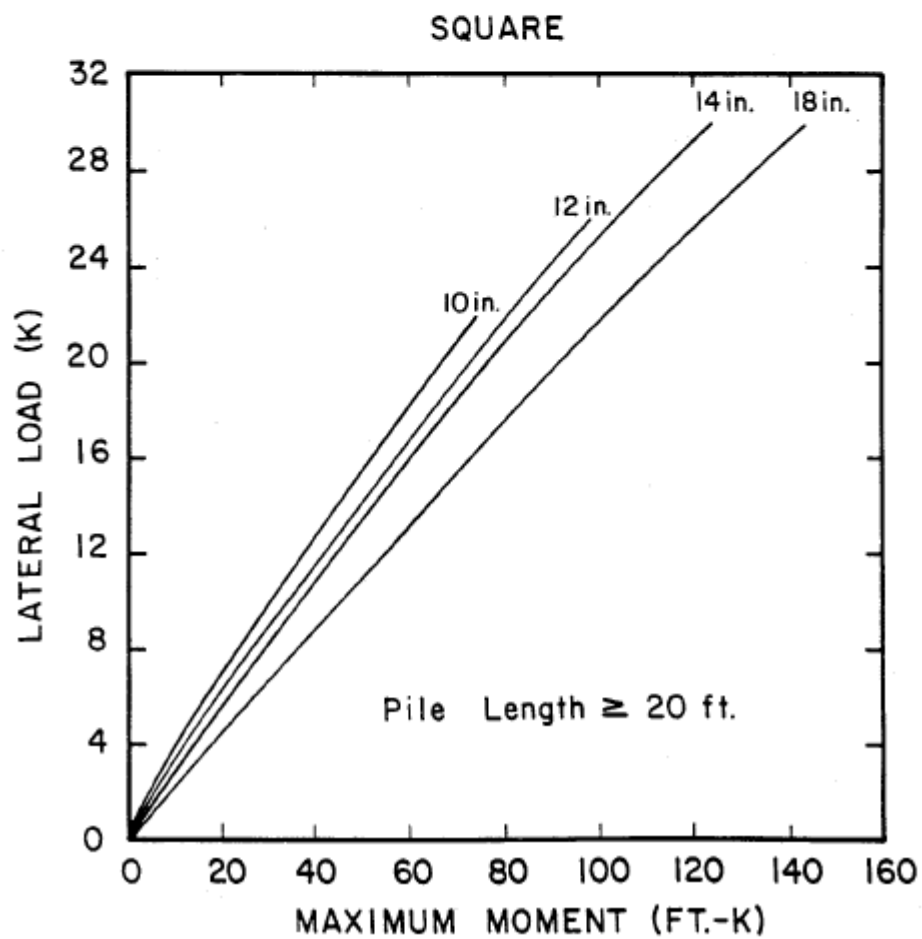
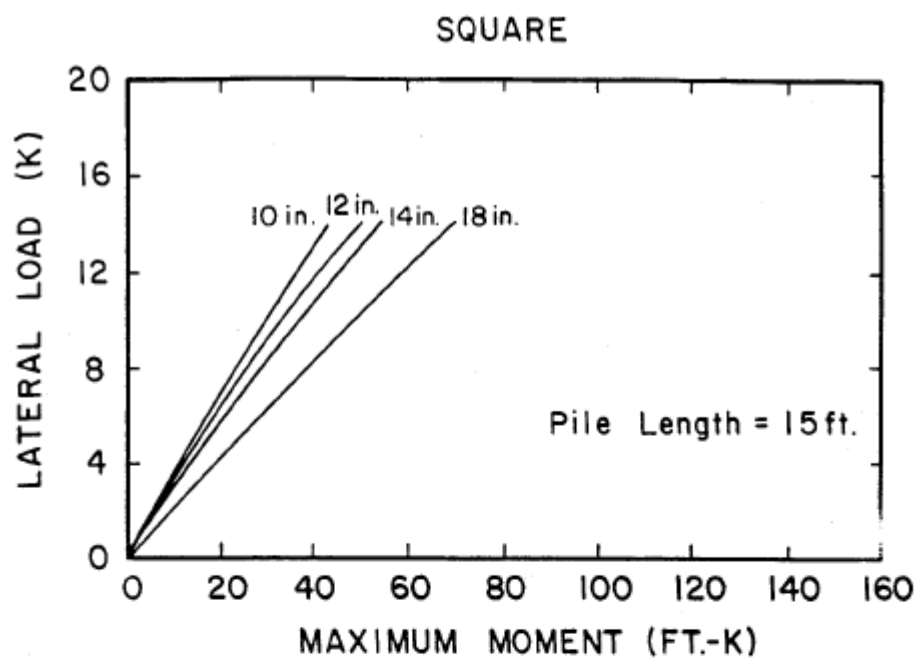


Figure 1.1.8-41 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 5

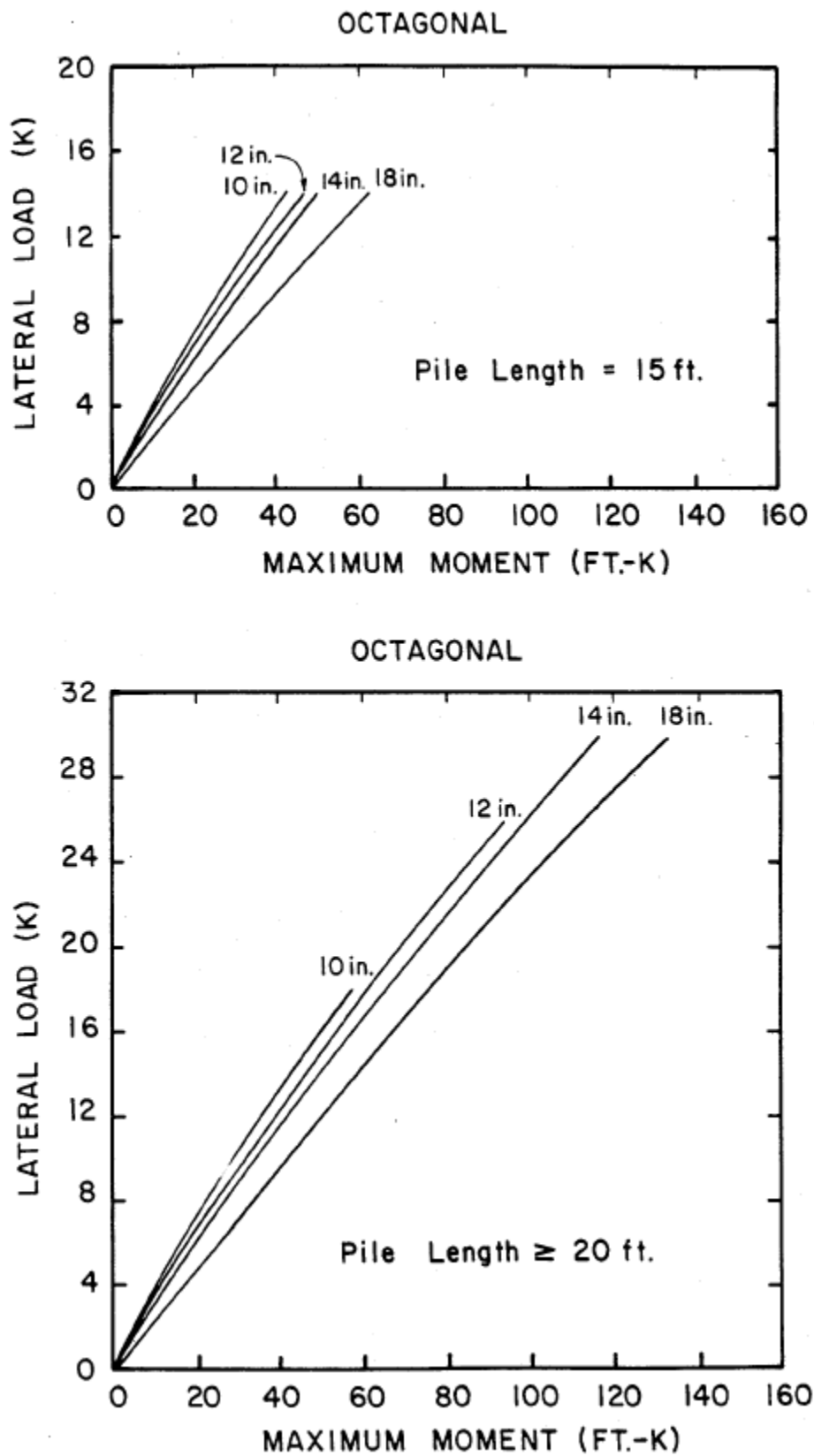


Figure 1.1.8-42 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 5

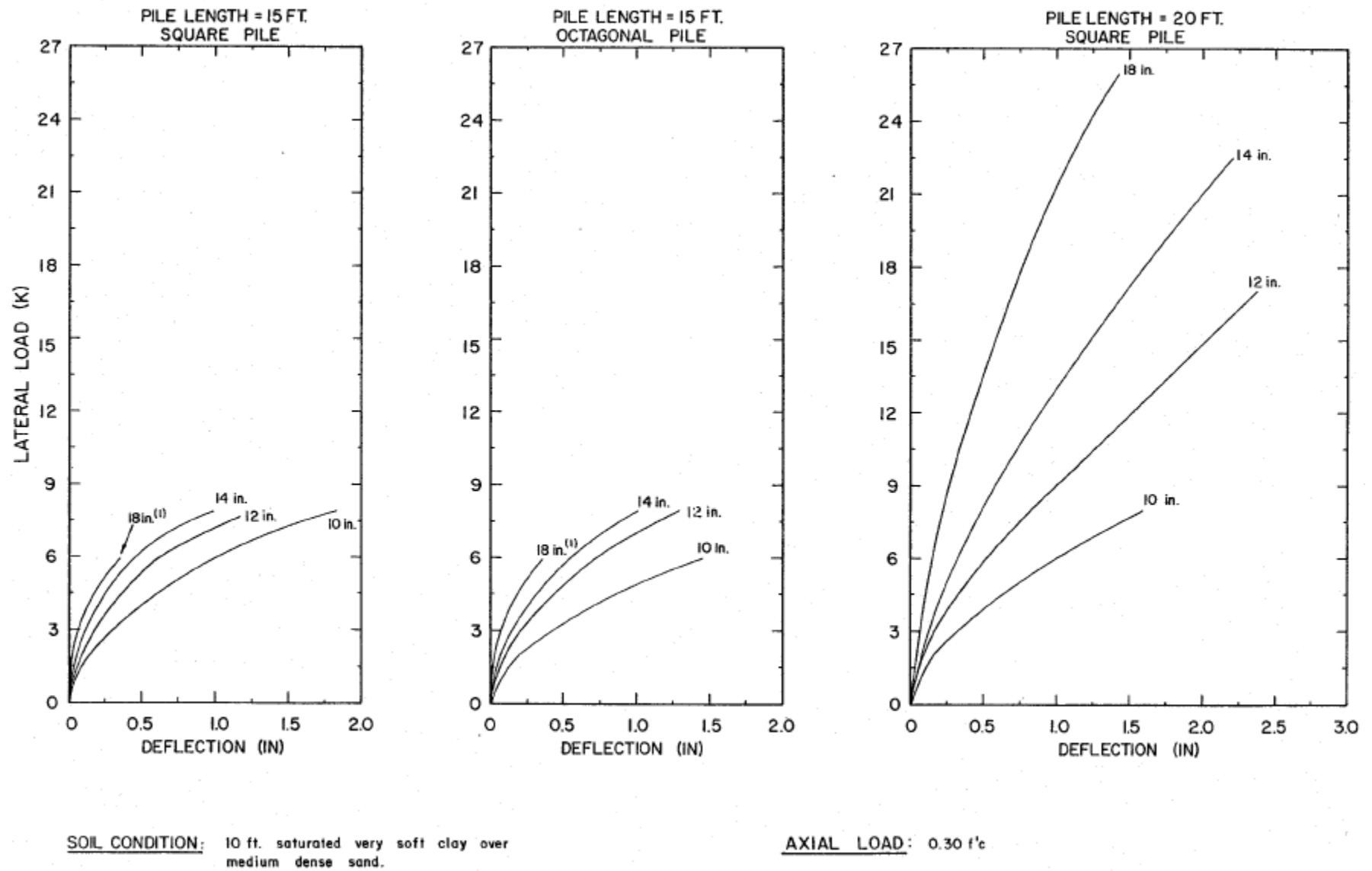


Figure 1.1.8-43 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 6

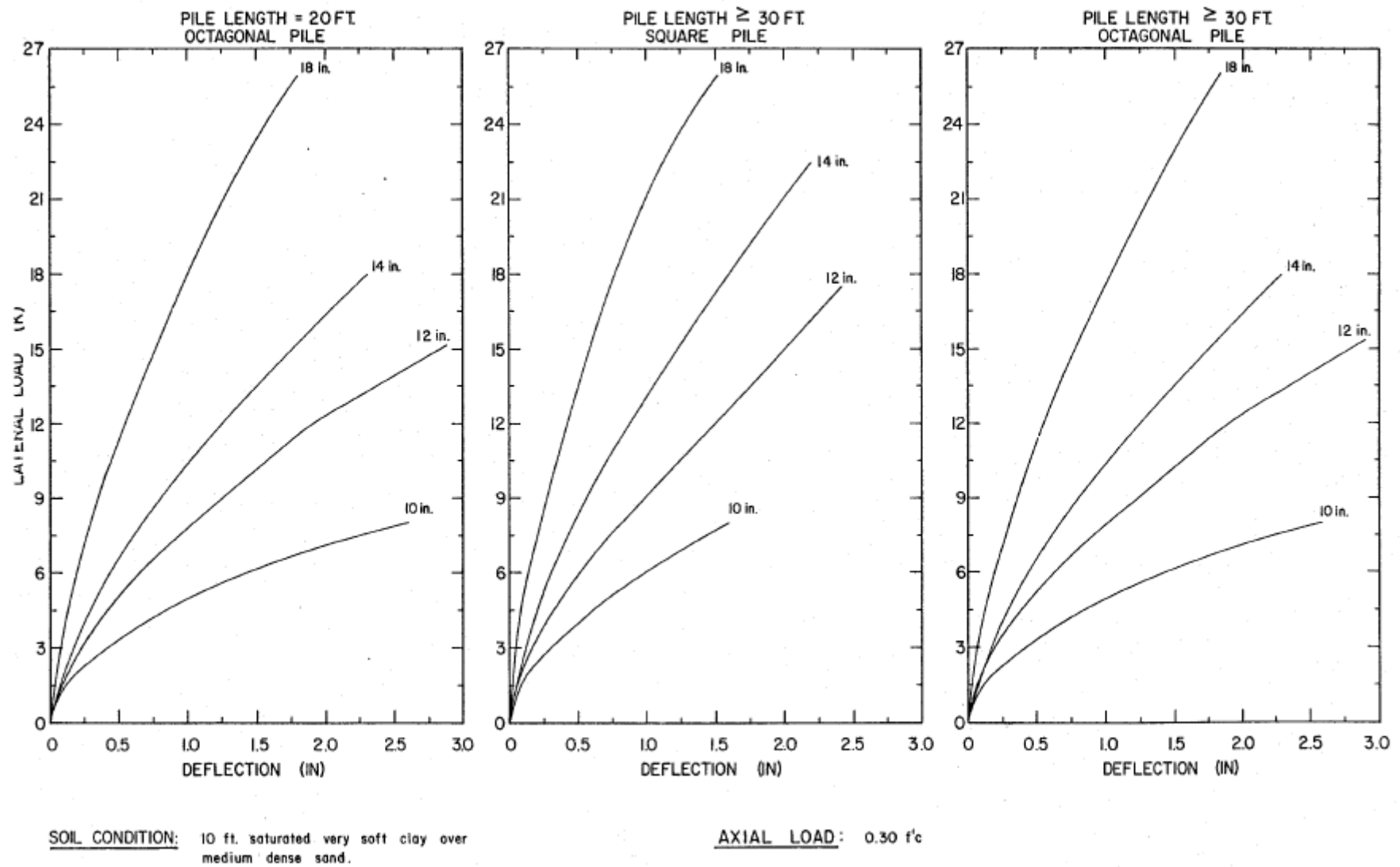


Figure 1.1.8-44 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 6

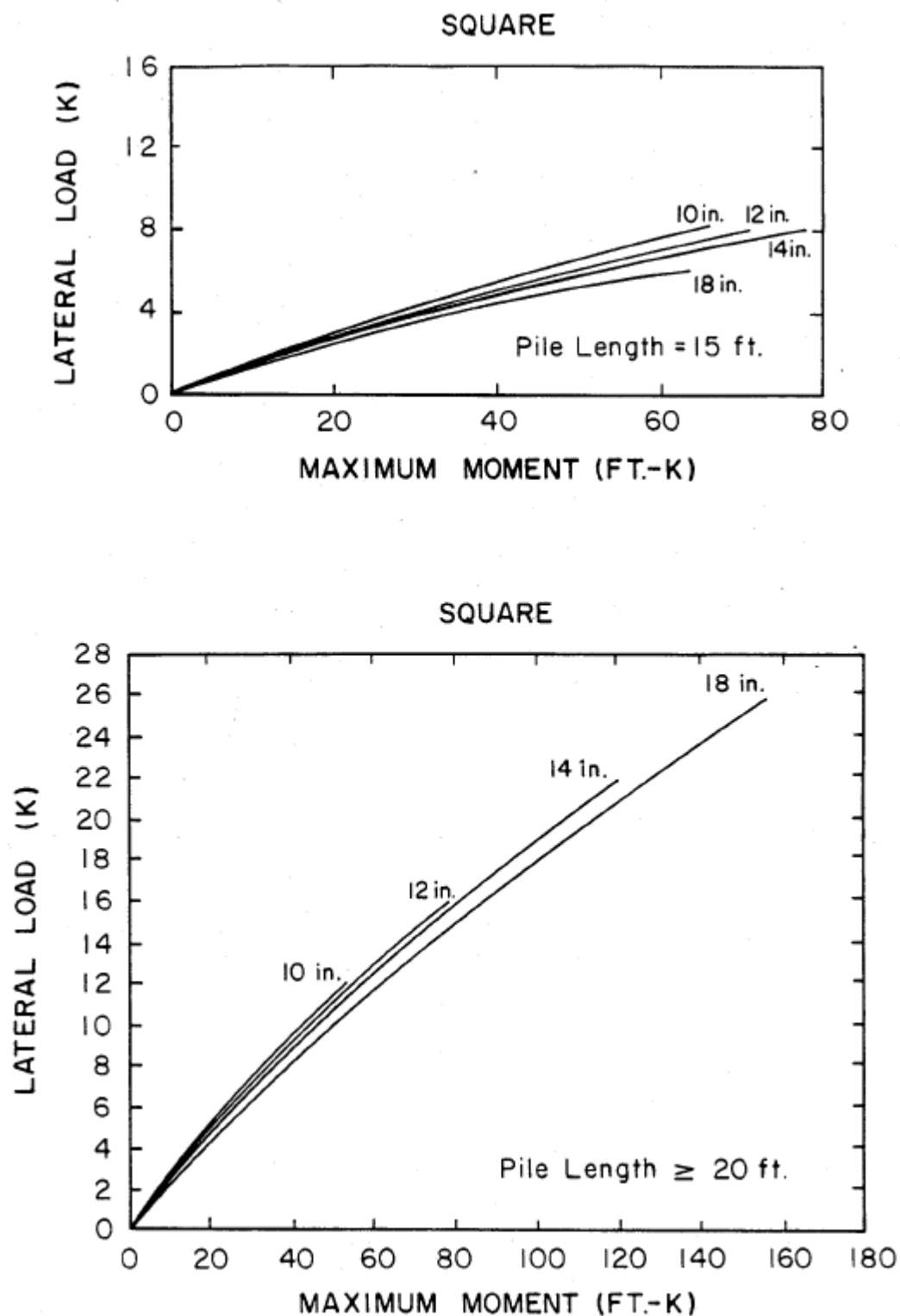


Figure 1.1.8-45 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 6

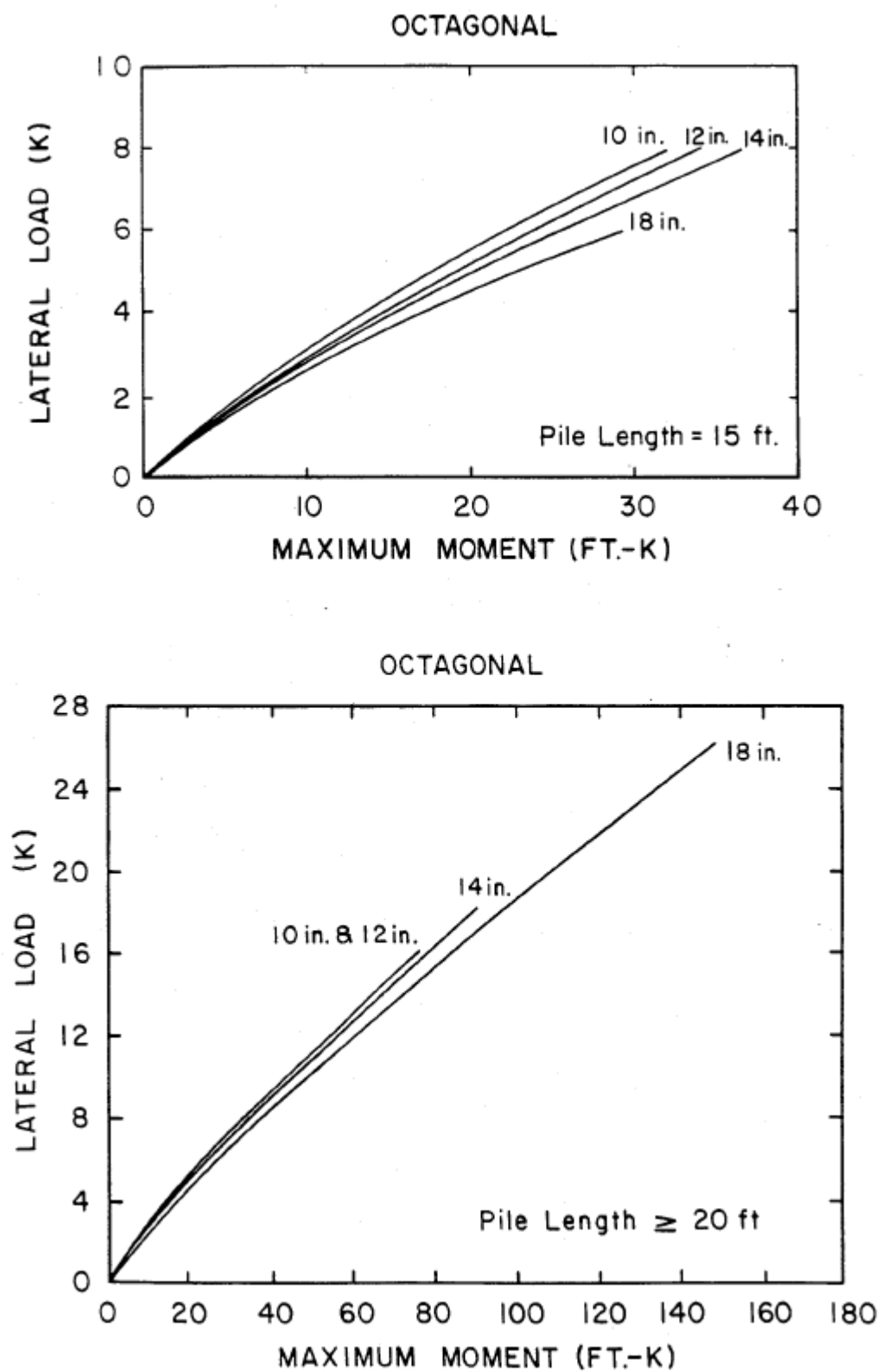
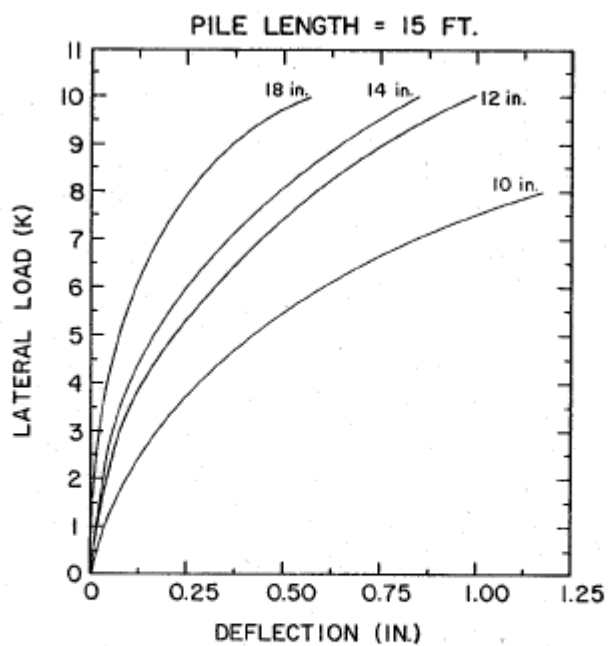


Figure 1.1.8-46 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 6



SOIL CONDITION: Soil Profile No. 7 — Saturated soft clay over 2 ft. soft or weathered rock.

REMARKS: Curves apply to both square and octagonal piles.

Figure 1.1.8-47 - Load Versus Deflection for Precast Concrete Pile, Soil Profile 7

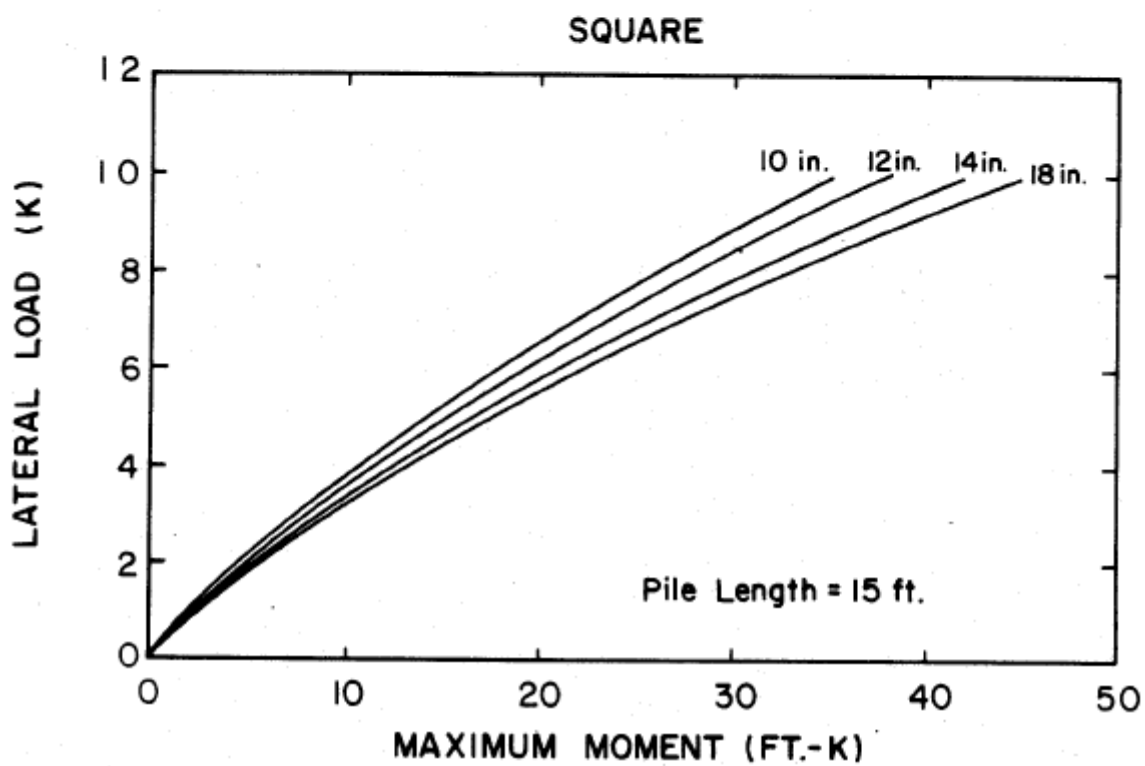
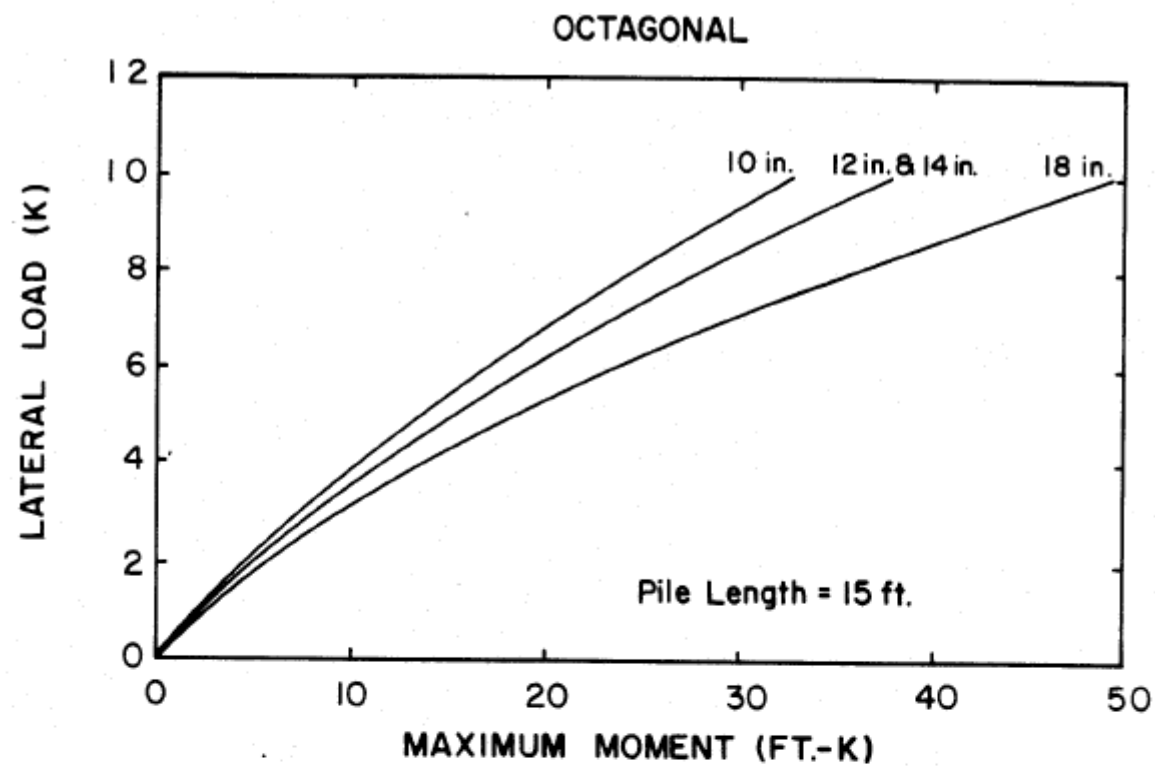


Figure 1.1.8-48 - Load Versus Maximum Moment for Precast Concrete Pile, Soil Profile 7

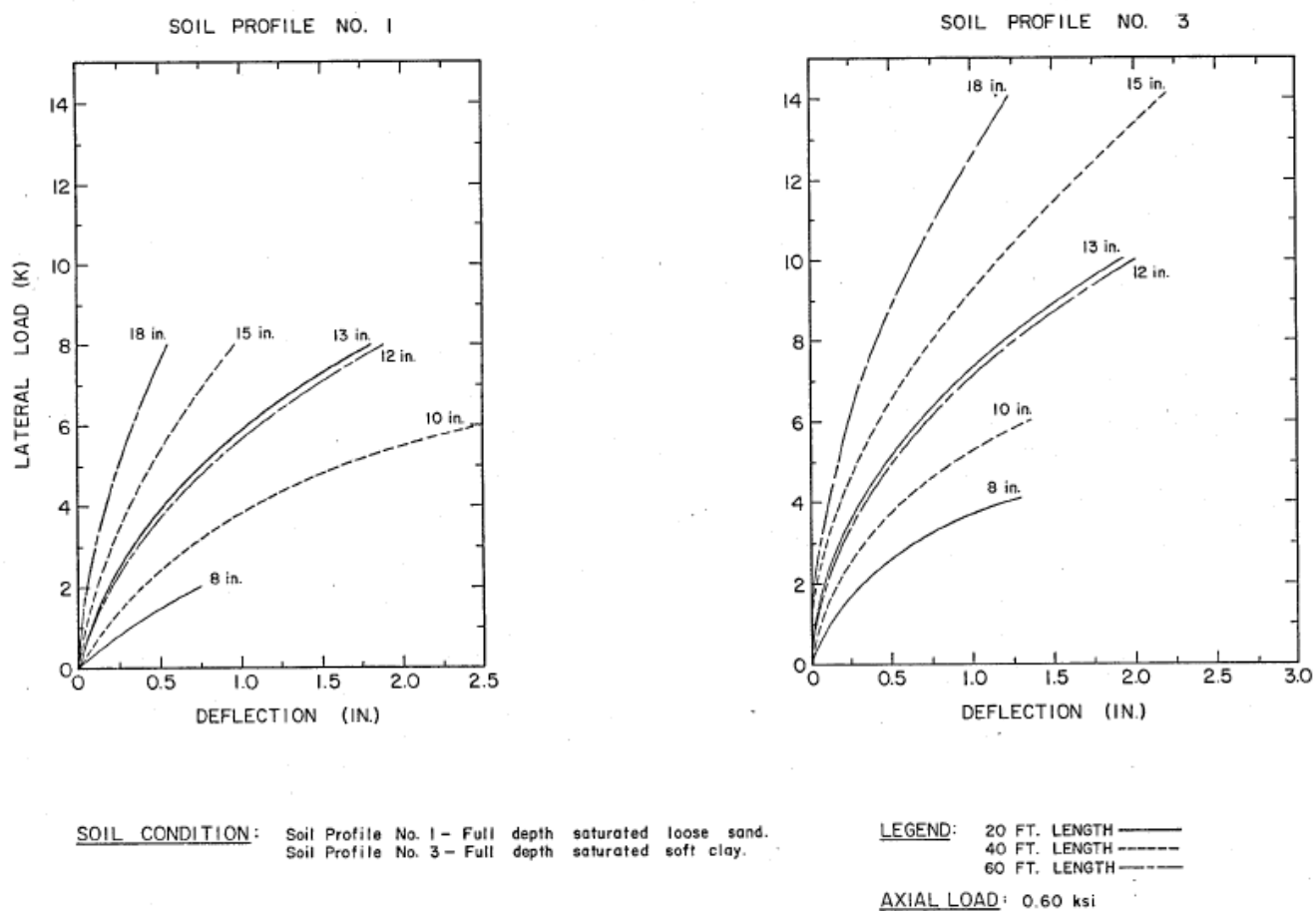


Figure 1.1.8-49 - Load Versus Deflection for Timber Pile, Soil Profile 1 and 3

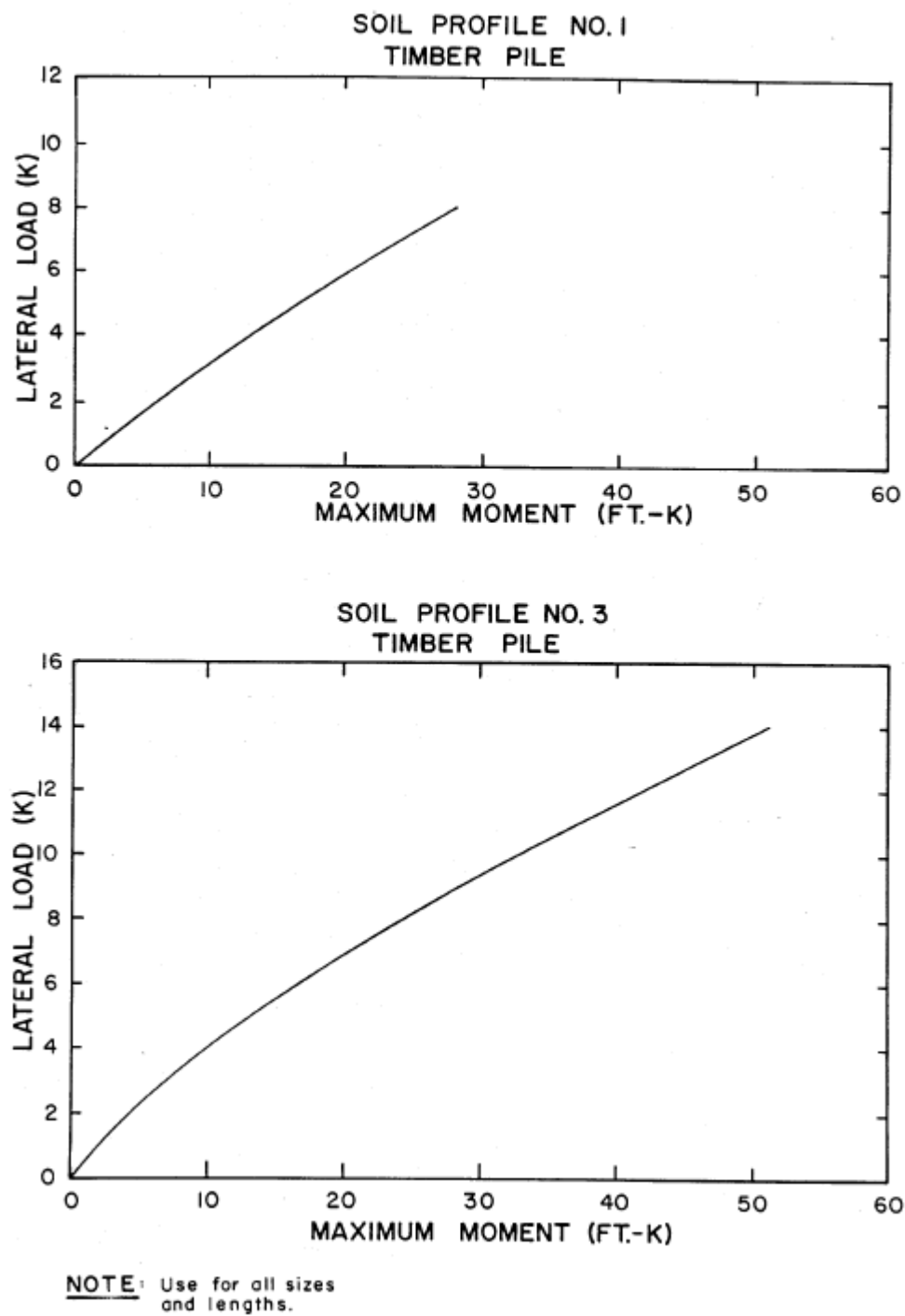


Figure 1.1.8-50 - Load Versus Maximum Moment for Timber Pile, Soil Profile 1 and 3

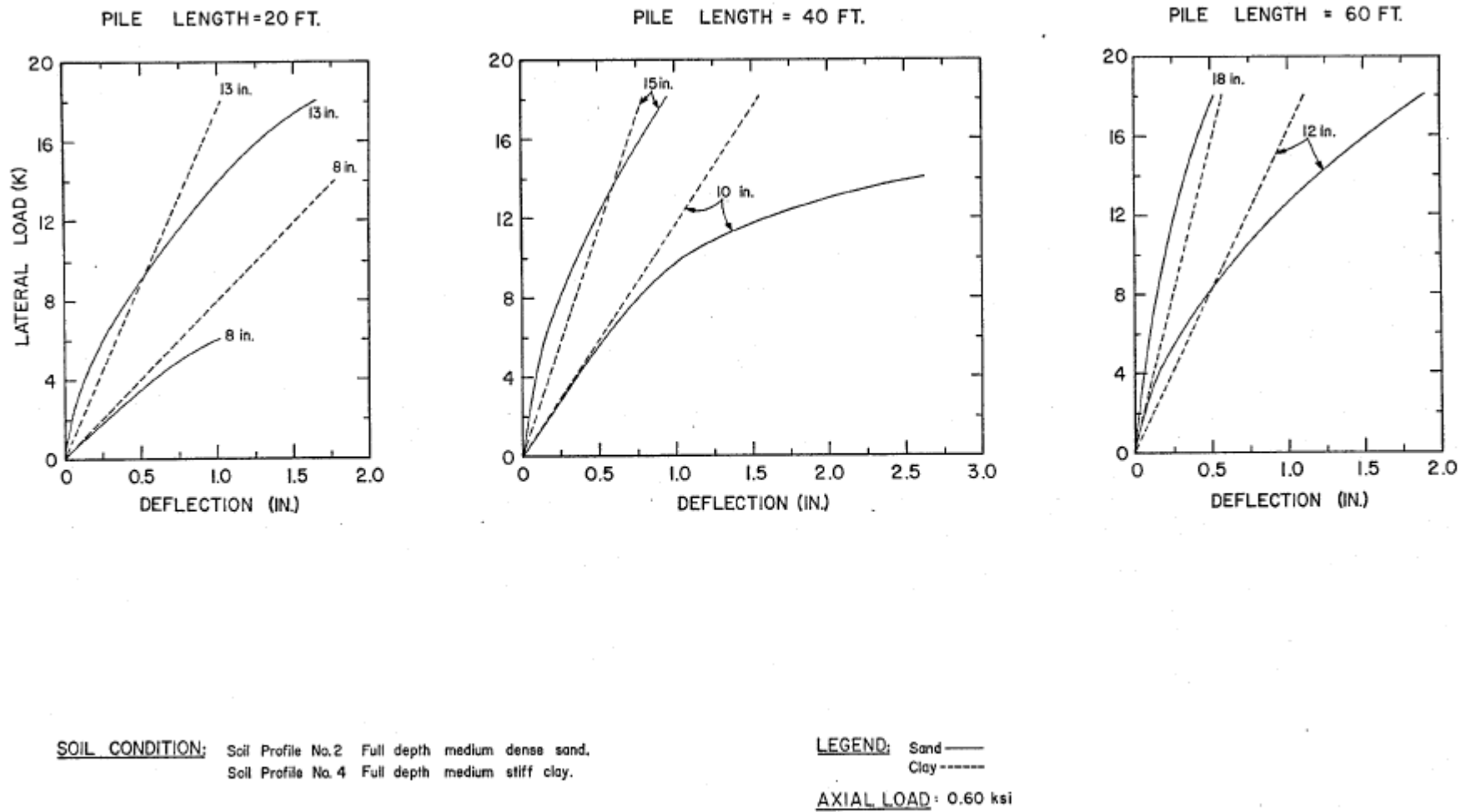


Figure 1.1.8-51 - Load Versus Deflection for Timber Pile, Soil Profile 2 and 4

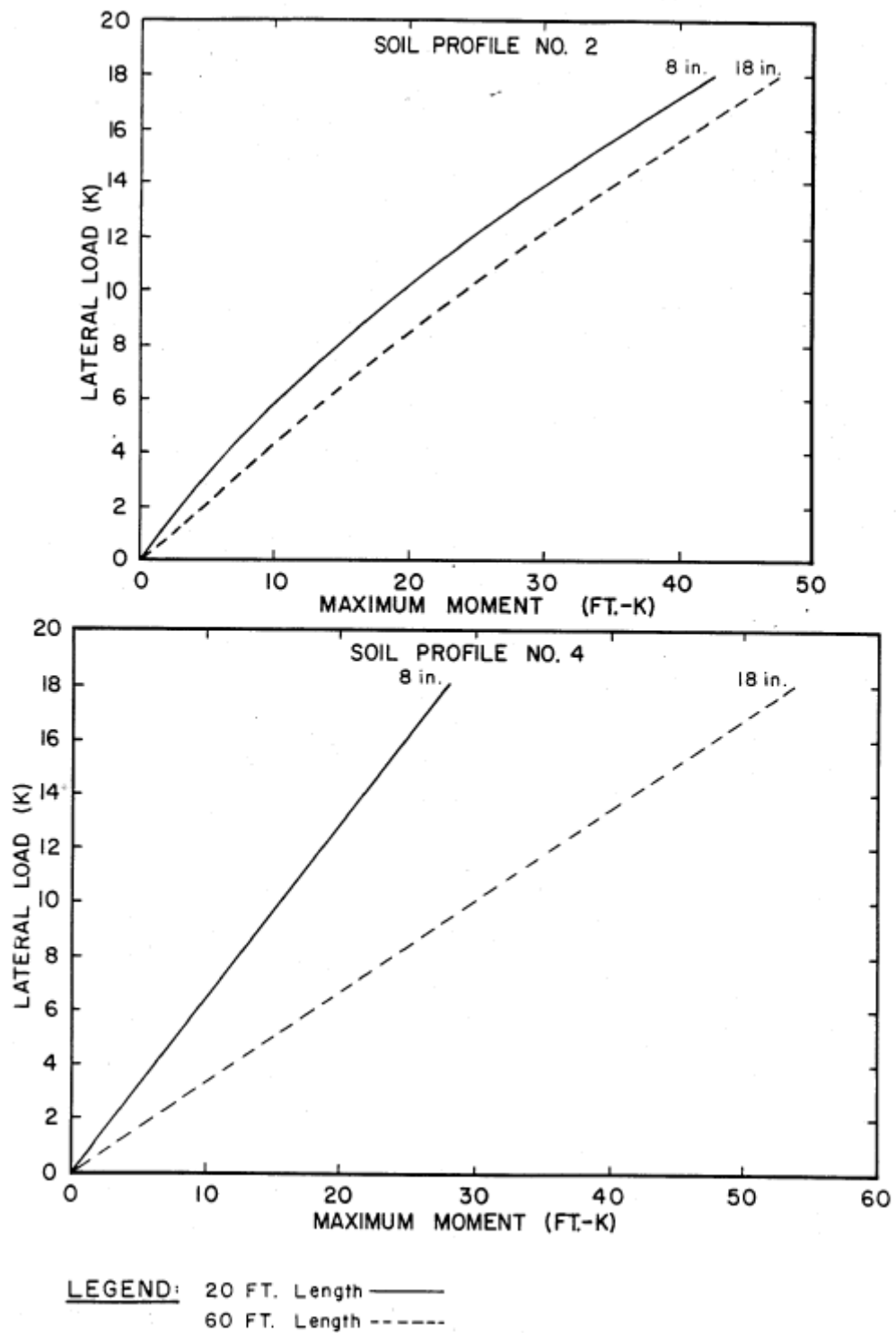
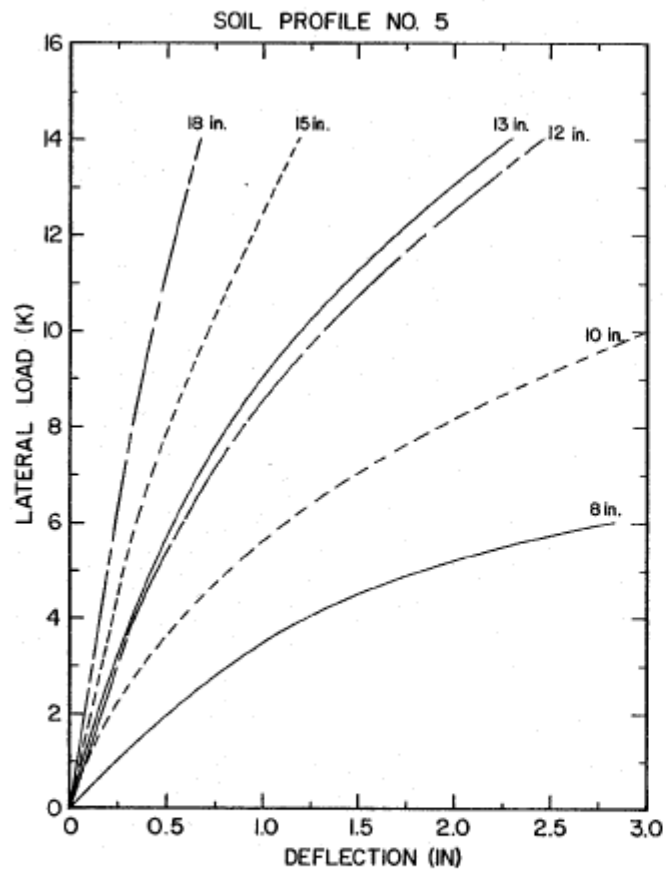
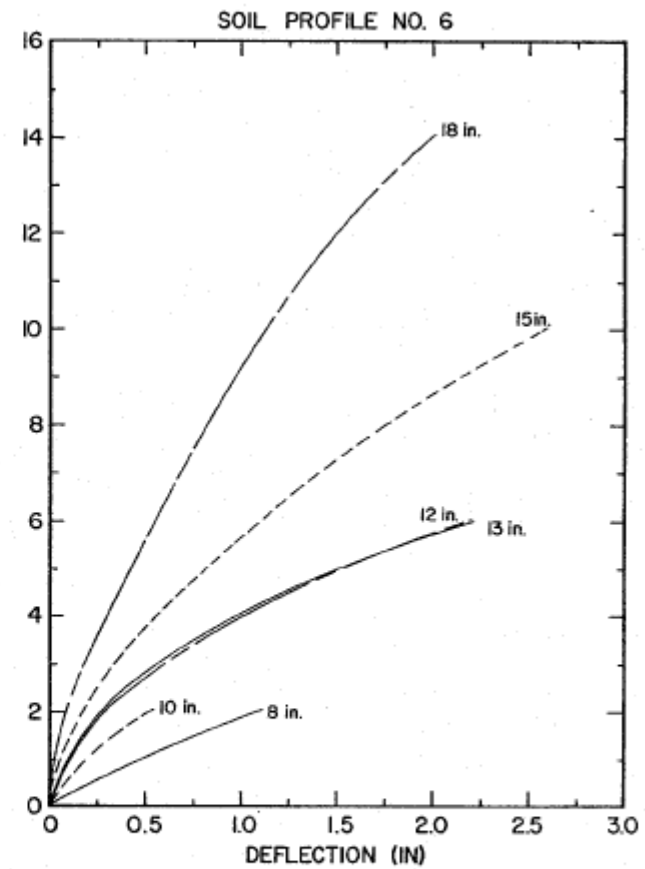


Figure 1.1.8-52 - Load Versus Maximum Moment for Timber Pile, Soil Profile 2 and 4



SOIL CONDITION: Soil Profile No. 5 - 10 FT. very loose sand overlaying saturated medium stiff clay.
 Soil Profile No. 6 - 10 FT. saturated very soft clay over medium dense sand.



LEGEND: 20 FT. Length ———
 40 FT. Length - - - - -
 60 FT. Length - - - - -

AXIAL LOAD: 0.60 ksi

Figure 1.1.8-53 - Load Versus Deflection for Timber Pile, Soil Profile 5 and 6

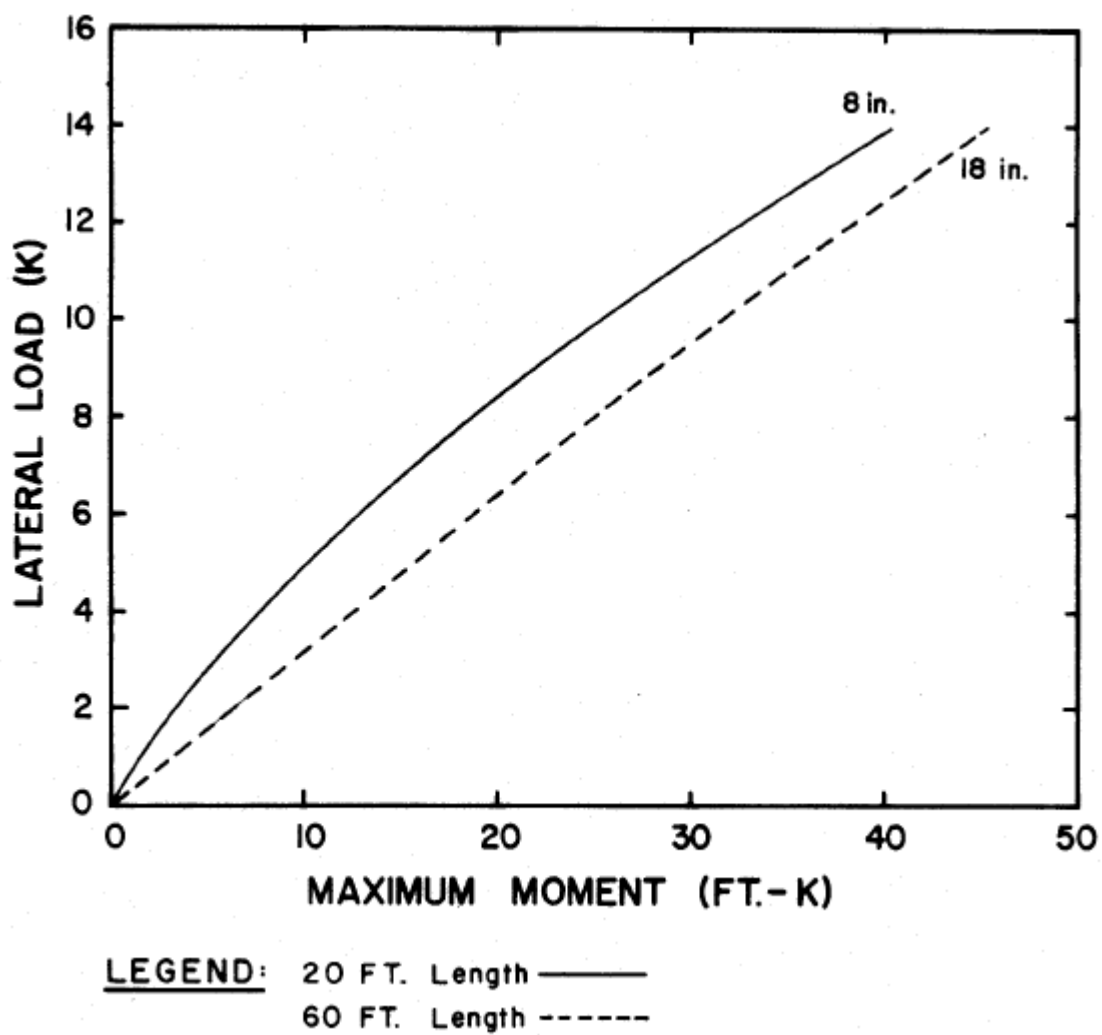
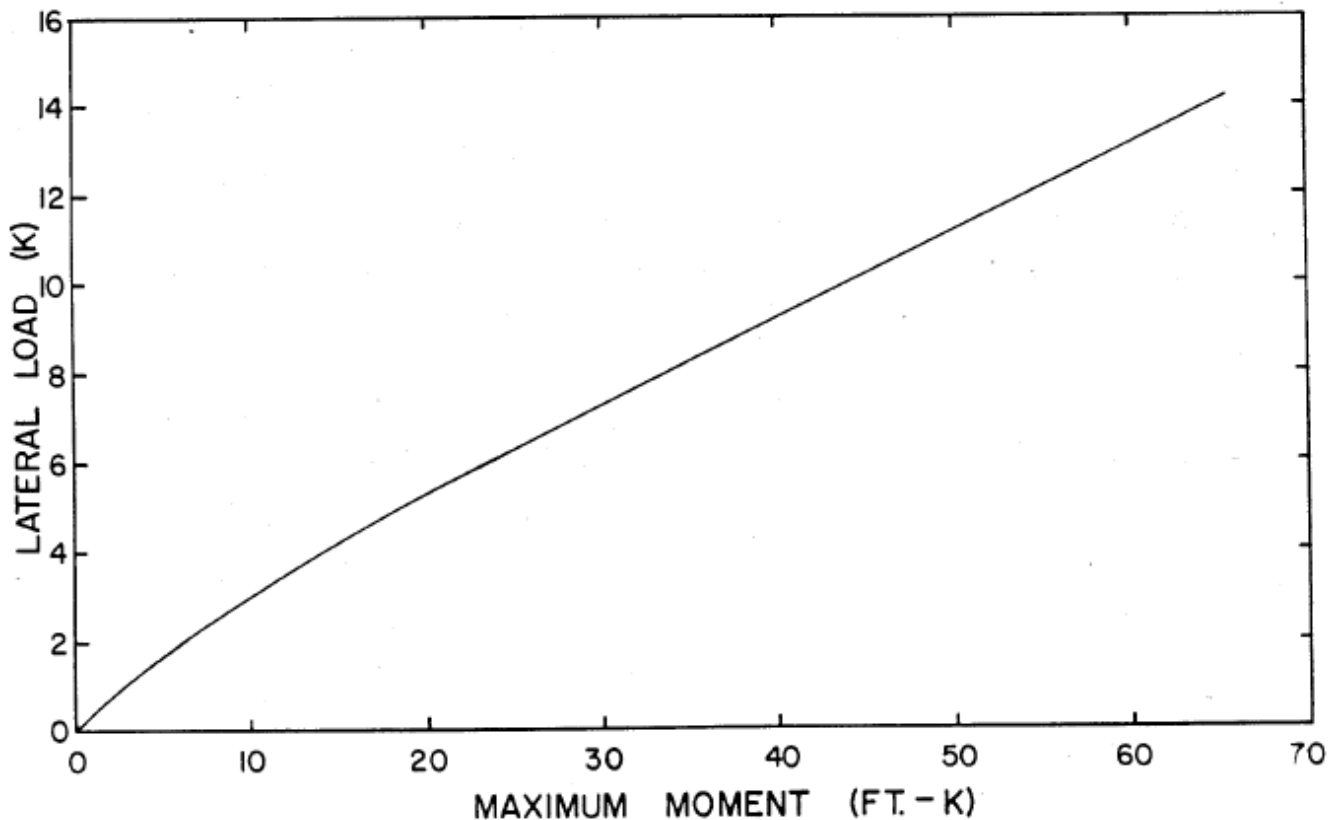


Figure 1.1.8-54 - Load Versus Maximum Moment for Timber Pile, Soil Profile 5



NOTE: Use for all sizes and lengths.

Figure 1.1.8-55 - Load Versus Maximum Moment for Timber Pile, Soil Profile 6

1.1.9 Additional Analyses

The lateral load versus deflection and lateral load versus moment curves can be used for the typical pier and abutment pile system. They provide conservative reasonable values for most design cases. Cases which shall be investigated independently include the following:

- (a) Soil conditions worse than shown on the soil profiles in Figure 1.1.6-1.
- (b) Single row of piles. A single row of piles performs similar to the free-head condition. Deflections taken from the curves may be considerably unconservative for a single row of piles.
- (c) Piles subject to lateral loads with large (greater than 1.0 in.) deflections. Fixities less than 50 percent may result from high lateral loads, resulting in unconservative deflections and possibly unconservative moments. Usually deflection limitations will prevent the application of such high loads.

If further increase of the lateral load resistance will allow substantial cost savings, additional analyses may also be performed for the following cases:

- (a) Soil conditions better than those shown on the soil profiles in Figure 1.1.6-1 in the Manual or borderline cases where soil conditions fall between those covered by those soil profiles.

- (b) Piles with small (less than 0.25 in.) deflections controlling the design - The fixity of the pile subject to small lateral loads and deflections will be closer to 100 percent. Deflections found on the curves may be unnecessarily conservative in this case.

Additional analyses shall be performed with the COM624G program following the procedures outlined in the program documentation [4,11]. For known C , ϕ and γ_T , other input parameters can be estimated based on recommendations within Reese's work [4]. Pile head fixities other than free of 100 percent fixed can be modeled with the following procedure:

- (a) For each lateral load, run the fixed-head case to obtain the fixed-head moment.
- (b) Apply the appropriate percentage of the fixed-head moment to the top of the pile, and run the free-head case. For example, for 50 percent fixity, 50 percent of the fixed-head moment would be applied to the top of the pile in the direction which would restrain movement. The program would then be run for the free-head case with the appropriate lateral load.

1.1.10 Lateral Resistance Design Procedure

Incorporate the lateral resistance of vertical piles into foundation design using the following procedure:

- (a) With the pile size and type selected for design and available geotechnical information, determine the moment in the pile caused by the factored lateral load by using the lateral load versus moment curves. This moment shall be included in the total factored moments used in the interactive equations for structural capacity found in AASHTO Sec. 10.4.7. A composite load factor shall be used when multiple lateral loads are used in a particular group loading.
- (b) Find the estimated horizontal deflection at the top of the pile caused by the lateral loads by using the lateral load versus deflection curves with the factored lateral loads.
- (c) If deflections exceed a value considered acceptable for the structure type, the following should be considered:
 - (1) Add batter piles to reduce design lateral load for pile.
 - (2) Revise pile size (preliminary design only)
 - (3) Add vertical piles to reduce the lateral load per pile.

2.1 DRILLED SHAFTS

2.1.1 General

The following sections list information on the development of design curves used to determine the lateral deflection and moment on drilled shafts when given the soil profile and a lateral load. The curves were originally developed for service load design. Since the method used was not dependant on the design method used (LRFD or Service Load Design), the curves can be used with any design methods as long as the specified units are consistent with the loads used. The design curves should be used as described in D10.7.3.12.2P.

2.1.2 Application

Drilled shafts are deep foundations formed by boring a cylindrical hole into soil and/or rock and filling the hole with concrete. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles or drilled piers. Similar to driven piles, drilled shafts transfer structural loads to bearing stratum well below the base of the structure. Drilled shafts are typically used to bypass soils having insufficient strength to carry the design loads.

Drilled shafts are classified according to their primary mechanism for deriving load resistance, and include floating shafts and end-bearing shafts in soil or rock. Occasionally, the base of the shaft is enlarged (i.e., belled or underreamed) to improve the load capacity of shafts bearing on less than desirable soils or to increase the uplift resistance of floating shafts.

2.1.3 Notations

The following notation shall apply for the design of drilled shaft foundations:

A	=	Coefficient used to define p-y curve for clay below water table by unified method (dim) (See 2.3.1.1.1.4)
A_c	=	Empirical coefficient for developing p-y curves for cyclic loading of stiff clay below water table (dim) (See 2.3.1.1.1.2)
$\} _c$	=	Empirical coefficient for developing p-y curves for cyclic loading of sand (dim) (See 2.3.1.1.1.5)
A_s	=	Empirical coefficient for developing p-y curves for static loading of stiff clay below water table (dim) (See 2.3.1.1.1.2)
$\} _s$	=	Empirical coefficient for developing p-y curves for static loading of sand (dim) (See 2.3.1.1.1.5)
B	=	Shaft diameter (ft.)
B_c	=	Empirical coefficient for developing p-y curves for cyclic loading of sand (dim) (See 2.3.1.1.1.5)
B_s	=	Empirical coefficient for developing p-y curves for static loading of sand (dim) (See 2.3.1.1.1.5)
c	=	Undrained shear strength (ksf)
C	=	Empirical coefficient for developing p-y curves for stiff clay above water table (dim) (See 2.3.1.1.1.3)
$\overline{C}$	=	Coefficient used in p-y curves for sand (dim) (See 2.3.1.1.1.5)
c'	=	Effective stress cohesive strength (ksf)
c_a	=	Average undrained shear strength over depth z (ksf)
D	=	Shaft length (ft.)
dz	=	Increment of shaft length (ft.)

e	=	Distance of P above ground surface for laterally loaded shaft (ft.)
E_c	=	Modulus of concrete shaft (ksf)
E_s	=	Soil modulus - secant to p-y curve (ksf)
E_{si}	=	Initial lateral soil modulus of subgrade reaction (kcf)
$(E_s)_{\max}$	=	Maximum lateral soil modulus of subgrade reaction (kcf) (See 2.3.1.1.1.4)
F	=	Empirical coefficient for developing p-y curves for clay below water table (dim) (See 2.3.1.1.1.4)
f'_c	=	Ultimate 28 day strength of concrete (psi)
i	=	Interval counter for load transfer analysis (dim)
I_c	=	Moment of inertia of shaft (ft. ⁴)
I_g	=	Gross moment of inertia of shaft section (ft. ⁴)
J	=	Empirical parameter for developing p-y curves for soft clay soil deposits (dim)
K	=	Lateral earth pressure coefficient (dim)
K_a	=	Active earth pressure coefficient (dim) ($[1 - \sin\phi']/[1 + \sin\phi']$)
k_c	=	Relative stiffness along soil/shaft interface due to cyclic lateral loading (kcf)
k_s	=	Relative stiffness along soil/shaft interface due to static loading (kcf)
K_o	=	At-rest lateral earth pressure coefficient equal to horizontal to vertical effective stress ratio (dim)
K_p	=	Passive earth pressure coefficient (dim) ($[1 + \sin\phi']/[1 - \sin\phi']$)
N	=	Number of cycles of cyclic lateral loading
NC	=	Normally consolidated (dim)
n_h	=	Coefficient of horizontal subgrade reaction (kcf)
P	=	Lateral load (k)
p_{cr}	=	Residual soil resistance for cyclic loading p-y curves by the Unified Method (kcf) (See 2.3.1.1.1.4)
p_m	=	Ultimate soil resistance per unit length for shaft in sand (kip/ft.)
p_r	=	Residual soil resistance for static loading p-y curves by the Unified Method (kcf) (See Figure 2.3.1.1.1.4-1)
p_s	=	Ultimate soil resistance per unit length for shaft in sand (kip/ft.)
p_{sd}	=	Ultimate soil resistance at depth for shaft in sand (kip/ft.) (See 2.3.1.1.1.5)
p_{st}	=	Ultimate soil resistance near ground surface for shaft in sand (k/ft.) (See 2.3.1.1.1.5)
p_u	=	Ultimate lateral soil resistance per unit length of shaft (kip/ft.)
P_{ULT}	=	Ultimate lateral load on shaft (kip)

y_c	=	Lateral deflection following N cycles of cyclic loading (ft.) (See Figure 2.3.1.1.1.3-2)
y_k, y_m, y_u	=	Specific deflection on p-y curve for sand (ft.) (See Figure 2.3.1.1.1.5-1)
y_p	=	Specific deflection on p-y curve for cyclic loading of stiff clay below water table (ft.) (See Figure 2.3.1.1.1.2-3)
y_s	=	Lateral deflection due to short term static load (ft.) (See Figure 2.3.1.1.1.3-2)
y_{50}	=	Lateral deflection at one-half p_u (ft.)
z	=	Depth along shaft from ground surface (ft.)
z_t	=	Depth to transition zone from wedge- to flow-type failure for a laterally loaded shaft (ft.) (See Figure 2.3.1.1.1.1-2)

The dimensional units provided with each notation are presented for illustration only to demonstrate a dimensionally correct combination of units for the design procedures and associated tables and figures presented herein. If other units are used, the dimensional correctness of the equations should be confirmed.

2.2 COM624 ANALYSES

Preliminary design curves have been developed for the six hypothetical subsurface profiles presented in Figure 2.2-1. Determine the static lateral capacity using the curves of lateral load versus deflection and lateral load versus absolute maximum moment for fixed and free head conditions. Use Figures 2.2-2 through 2.2-8 to determine load-deflection behavior for each of the subsurface profiles. Figures 2.2-7 and 2.2-9 provide relationships to determine the maximum moment for the loose sand and soft clay profiles, respectively, which represent the two profiles providing the highest level of moment development. Use linear interpolation for cases involving intermediate shaft dimensions or degrees of head fixity. For subsurface conditions not depicted in the figures, consider performing COM624 analyses incorporating the actual subsurface ground conditions.

Comments regarding important aspects of the analyses are summarized in Tables 2.2-1 and 2.2-2. The shaft sections were assumed to be unreinforced, so the concrete modulus of elasticity (E_c) and gross moment of inertia (I_g) were specified to determine the foundation stiffness ($E_c I_c$). The stiffness of a reinforced concrete section is dependent on the magnitude of the bending stress and amount of reinforcement; thus, the bending stiffness varies along the length of the shaft and with applied loads. However, use of the stiffness for the gross concrete section ($E_c I_g$), where E_c is approximated as $57000\sqrt{f'_c}$, is sufficiently accurate in most instances. When more refined analysis is required, computer methods can be employed to develop interaction diagrams for the shaft and establish effective stiffness at the appropriate stress levels.

In all cases, the option to have p-y curves generated internally was selected. The reader should be aware of the techniques used to formulate the soil responses and recognize their limitations, as described in Section 2.3.1.1.1. Section 2.3.1.1.1 was not converted to SI units because of the large number of empirical equations which are units dependant.

Representative combinations of soil properties for the subsurface profiles were estimated, and values for the strain at 50 percent stress level (E_{50}) and constant coefficient of subgrade reaction (n_h in $E_s = n_h z$) chosen. The values were based on recommendations in 2.3.1.1.1.1 through 2.3.1.1.1.5 for E_{50} , and n_h .

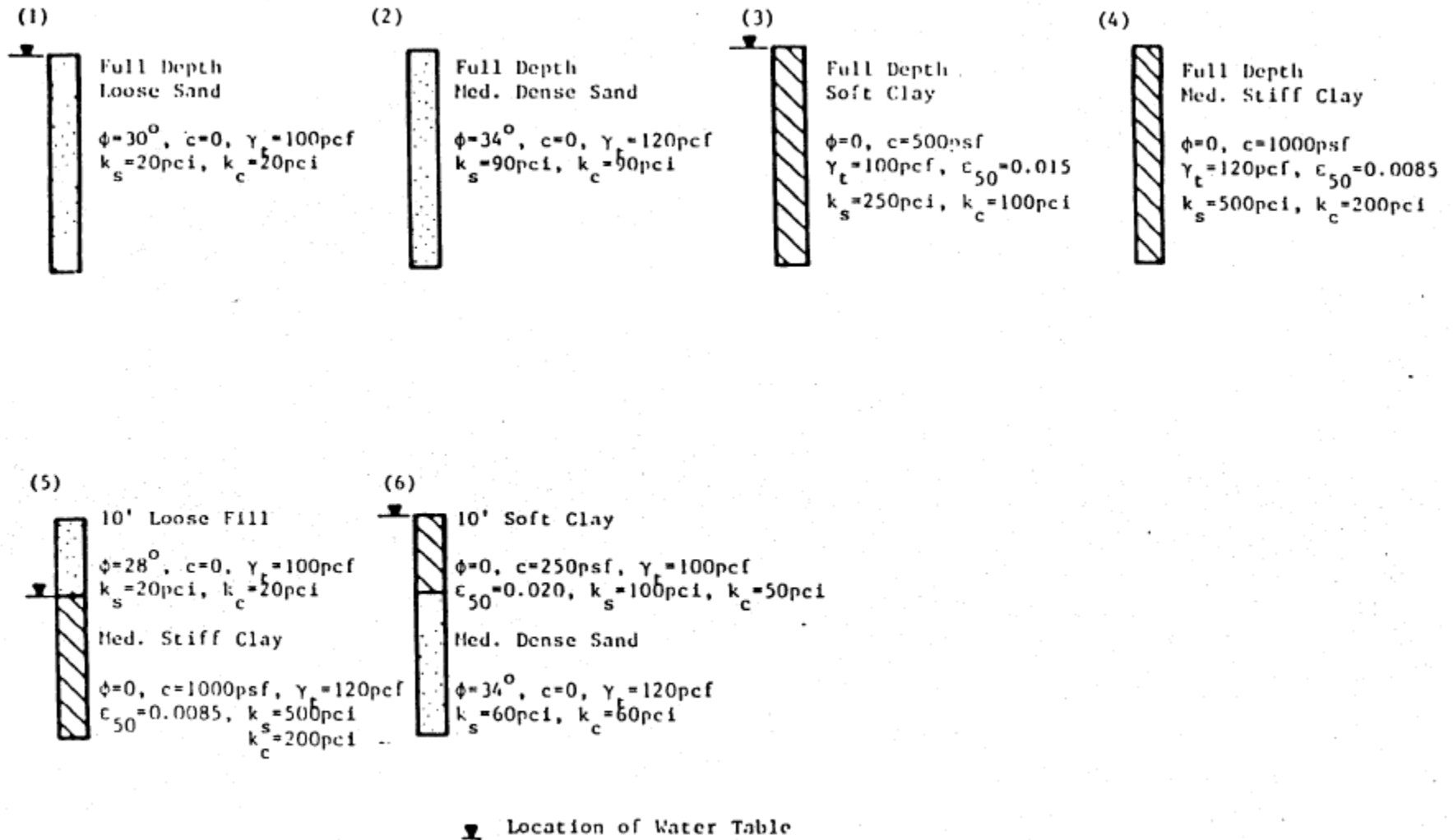


Figure 2.2-1 - Soil Profiles for COM624 Analyses

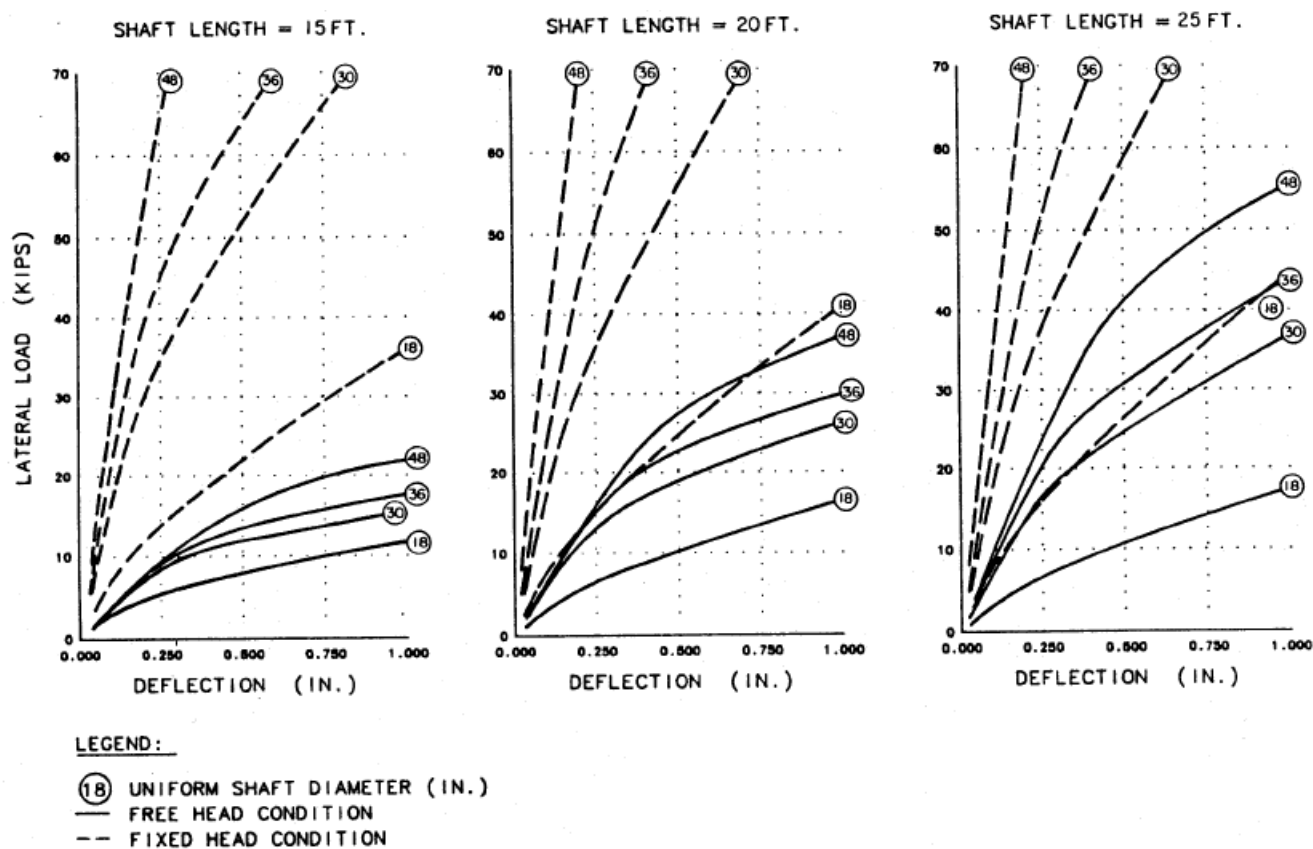


Figure 2.2-2A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 1

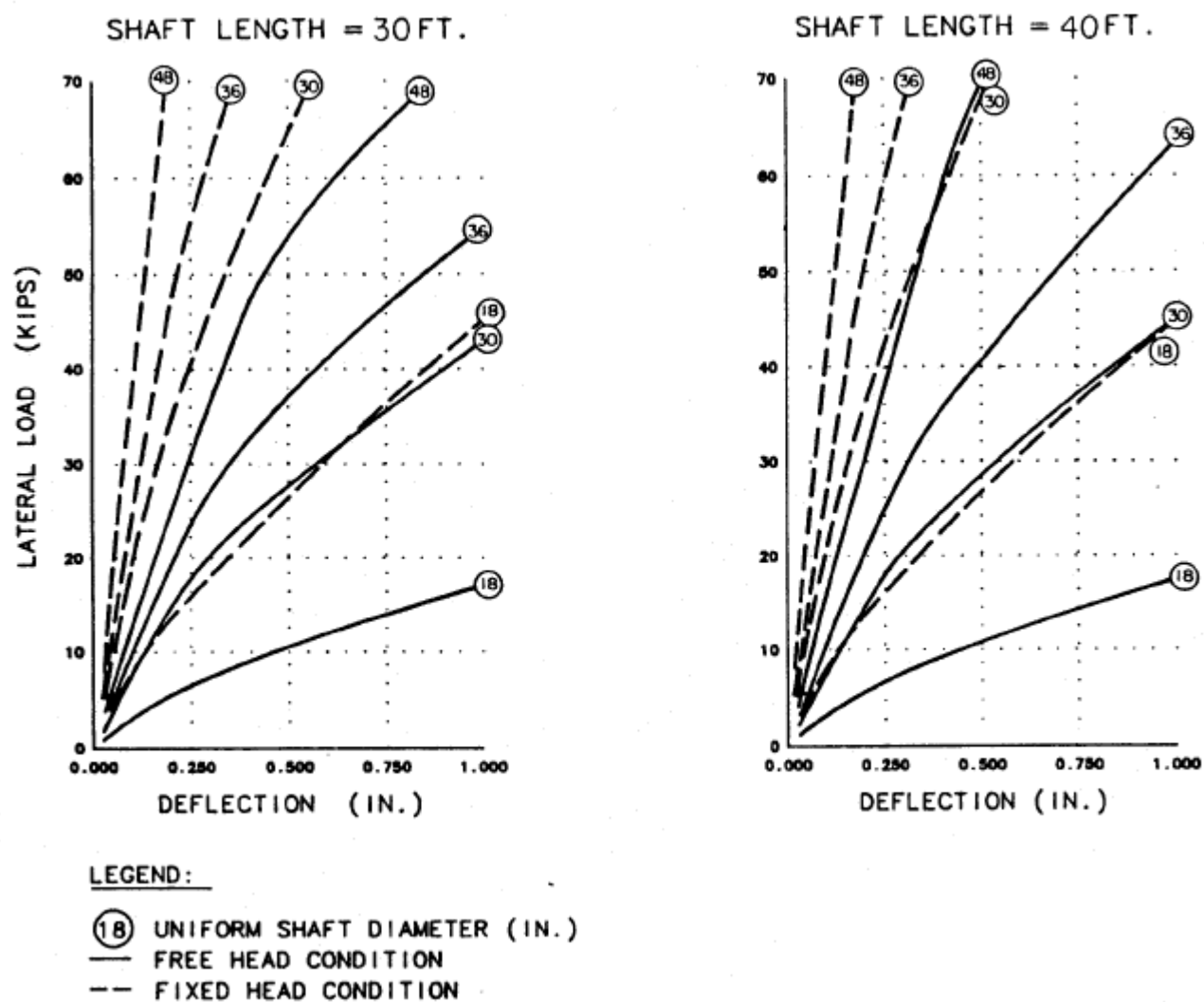


Figure 2.2-2B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 1 (Continued)

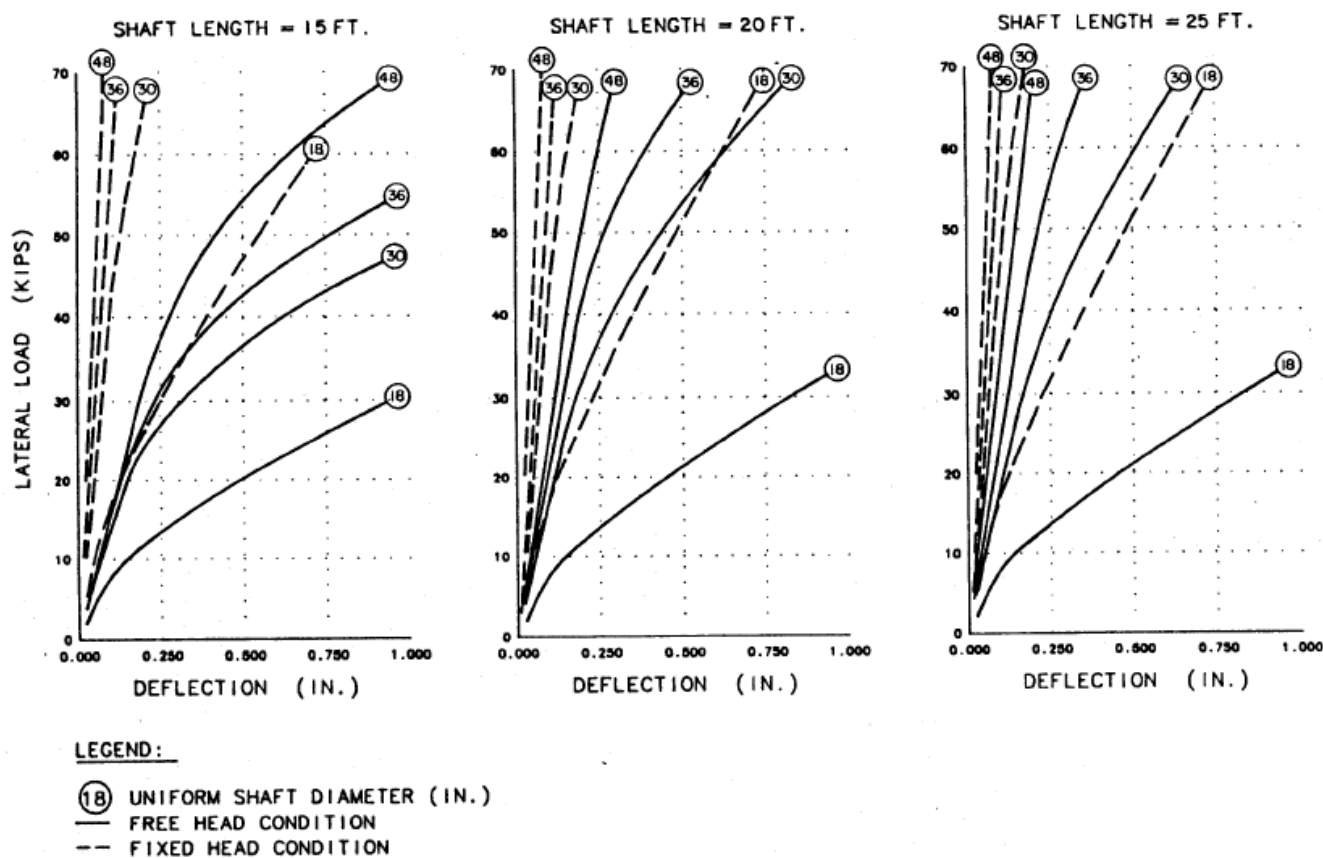


Figure 2.2-3A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 2

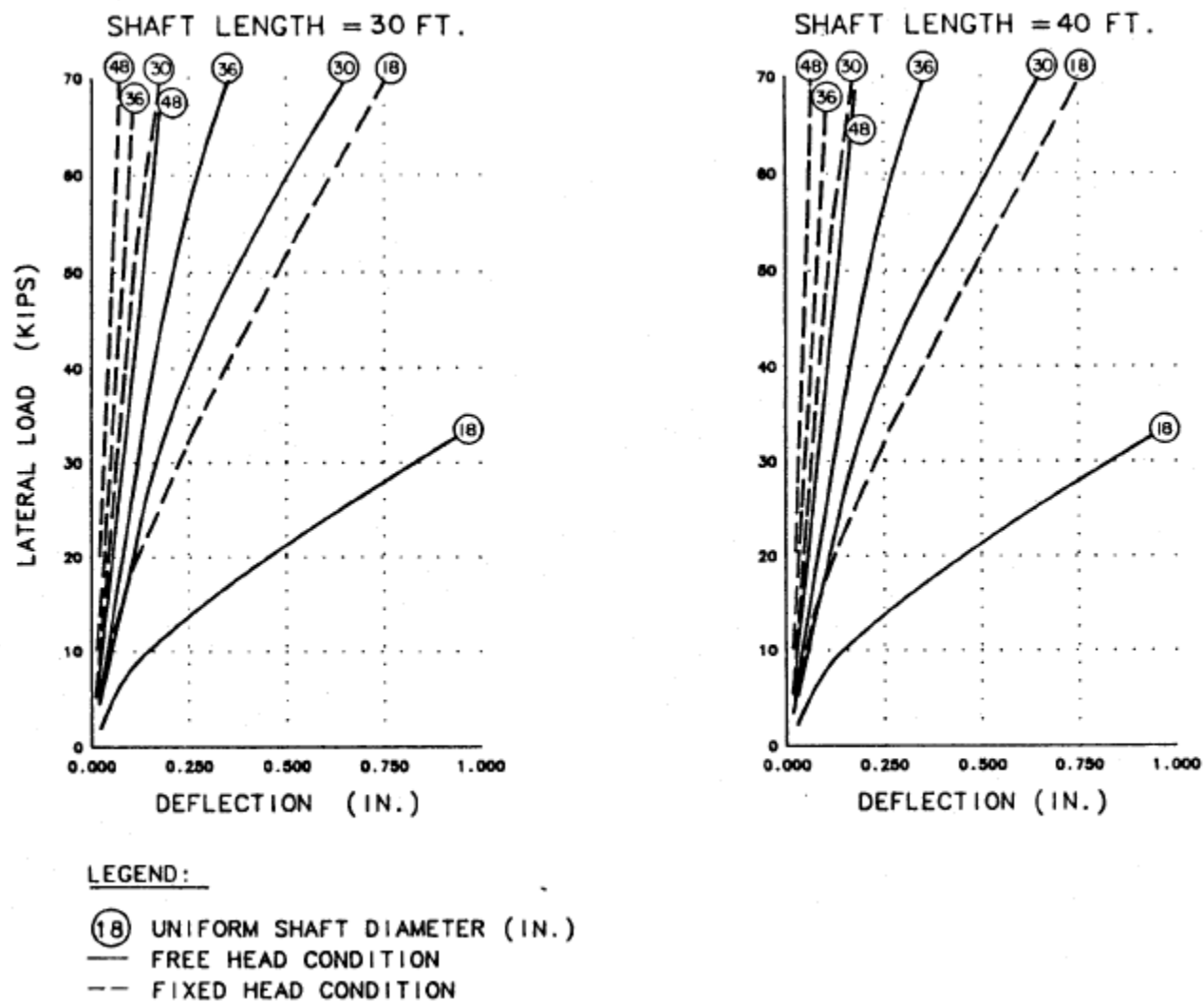


Figure 2.2-3B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 2 (Continued)

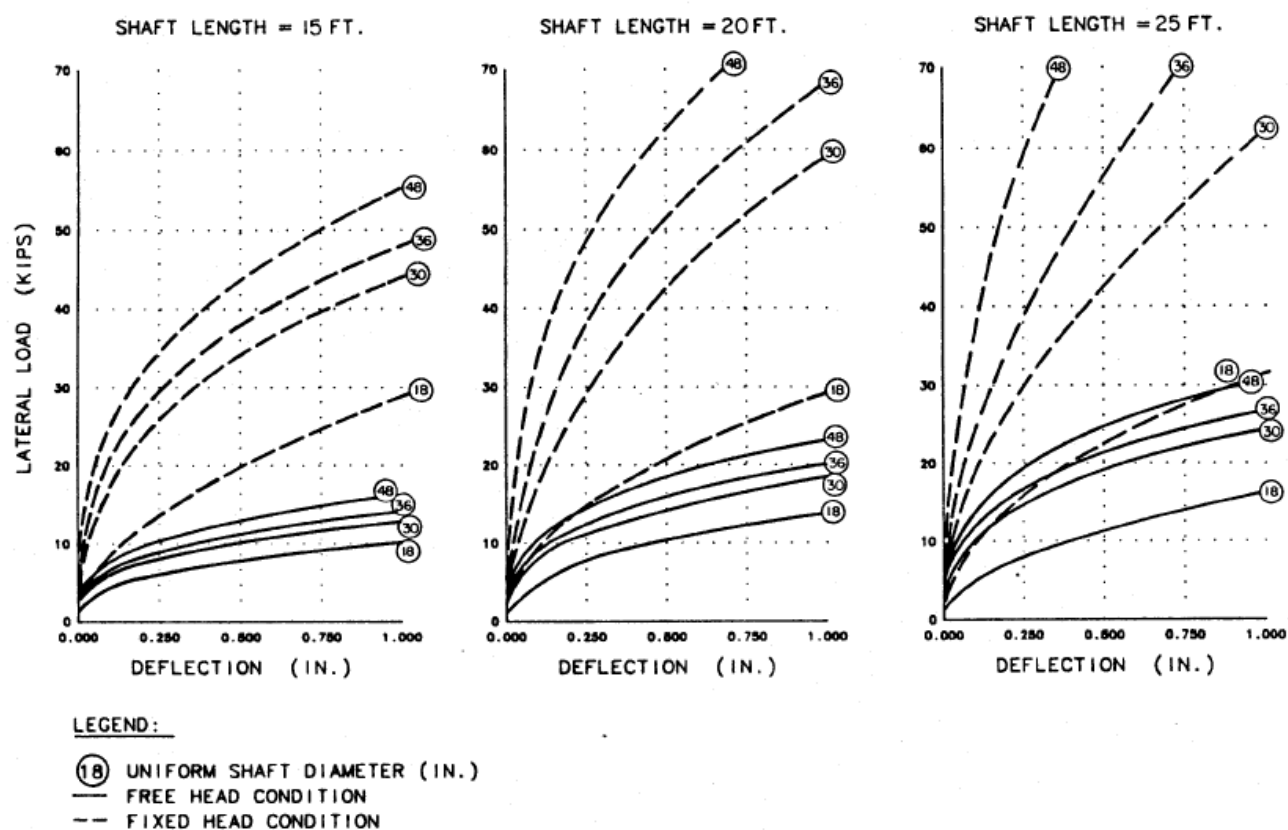


Figure 2.2-4A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 3

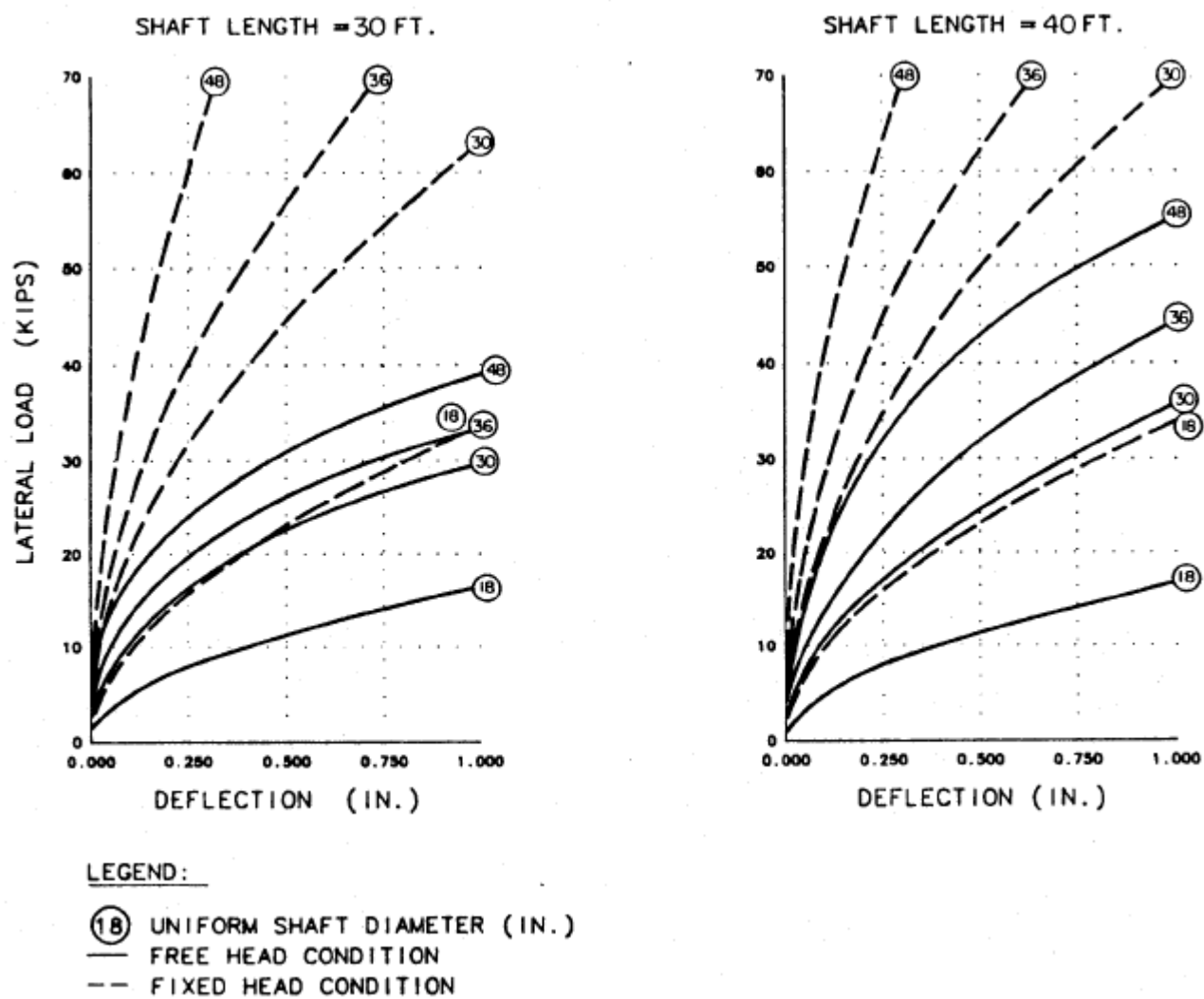


Figure 2.2-4B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 3 (Continued)

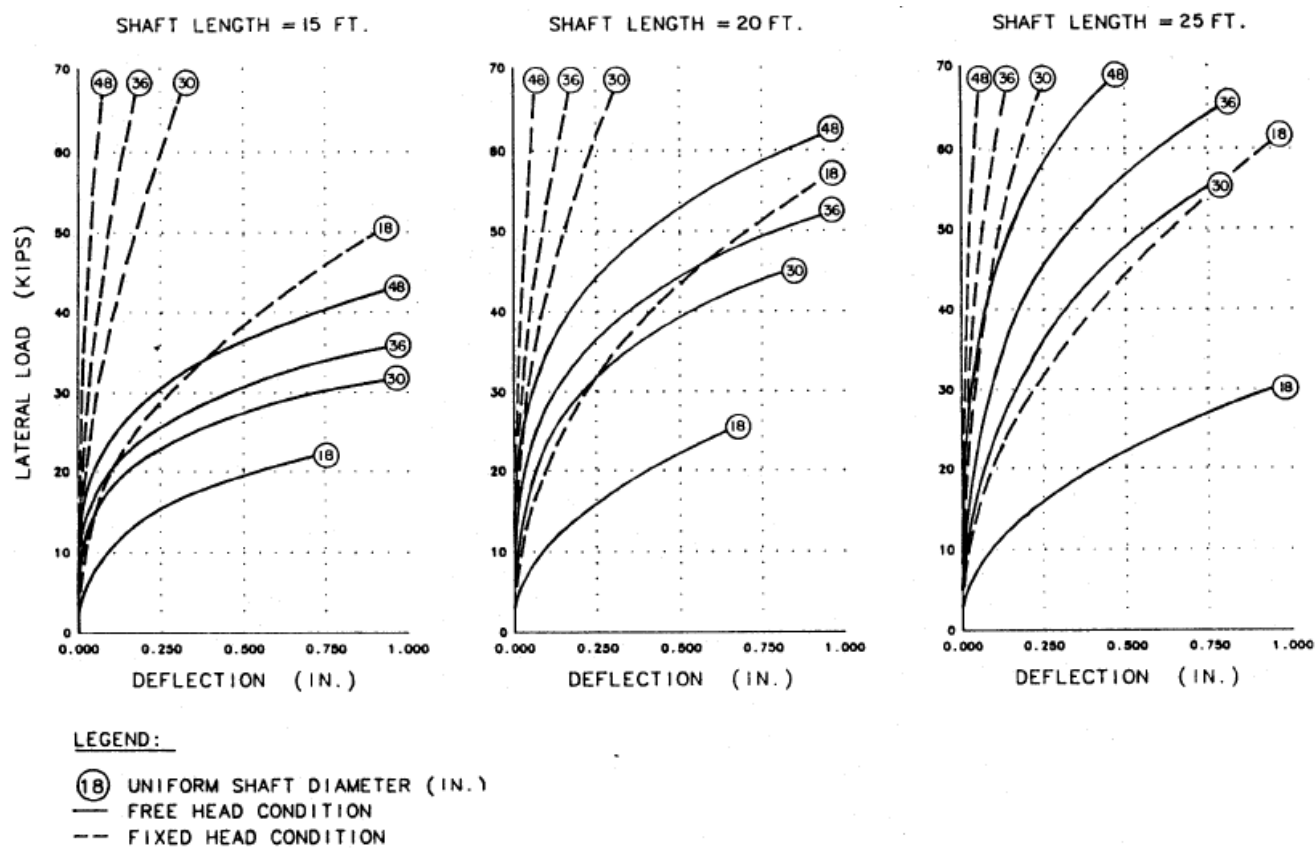


Figure 2.2-5A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 4

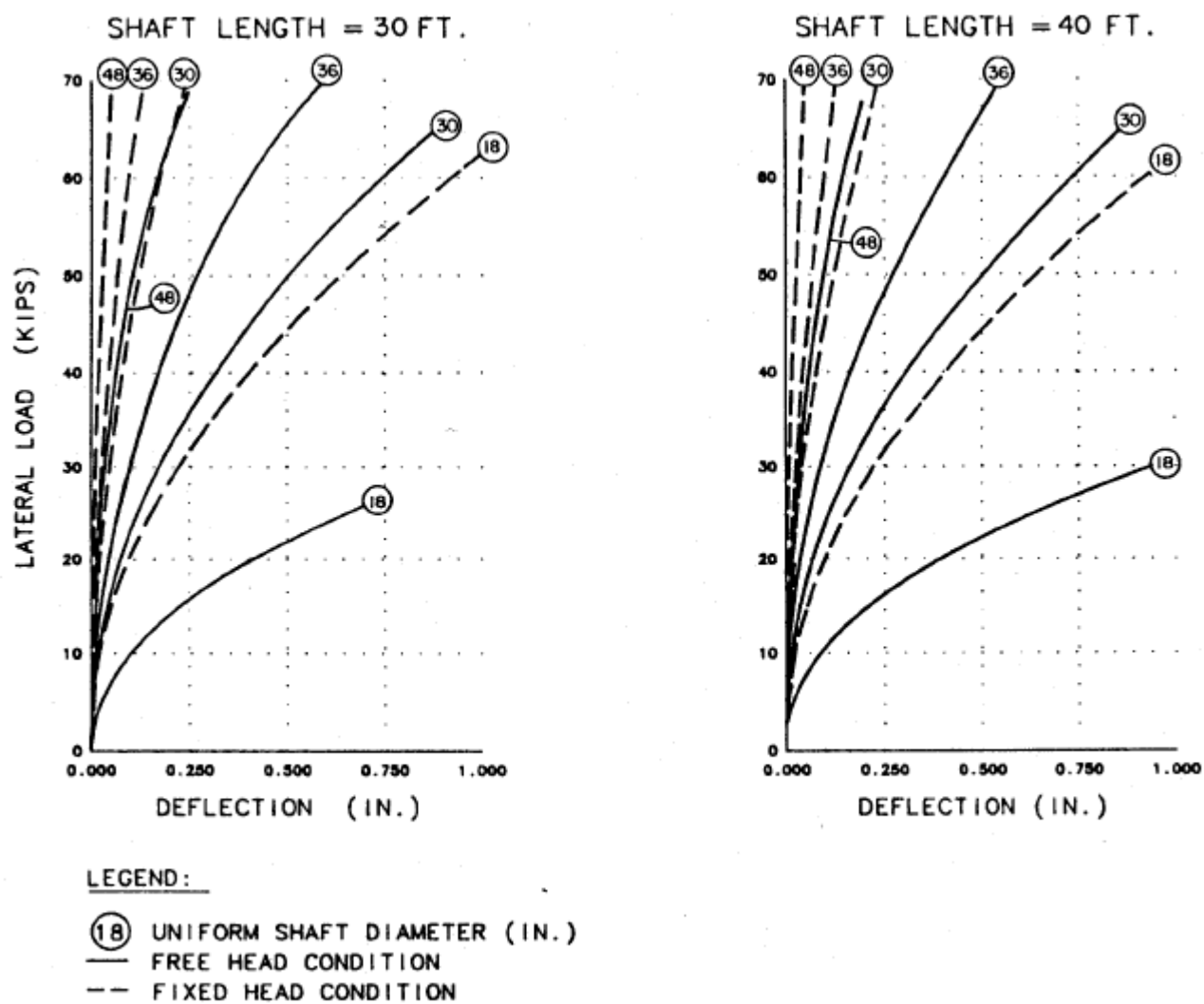


Figure 2.2-5B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 4 (Continued)

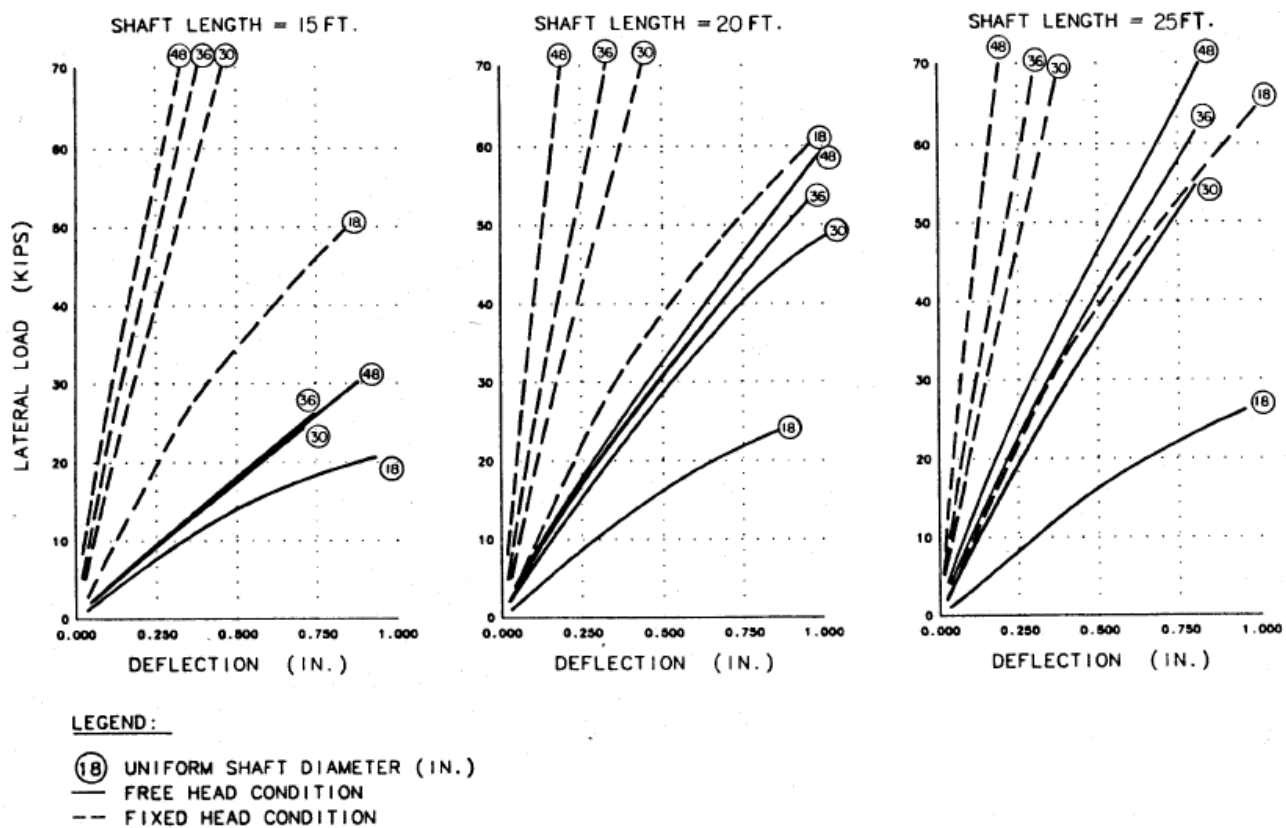


Figure 2.2-6A - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 5

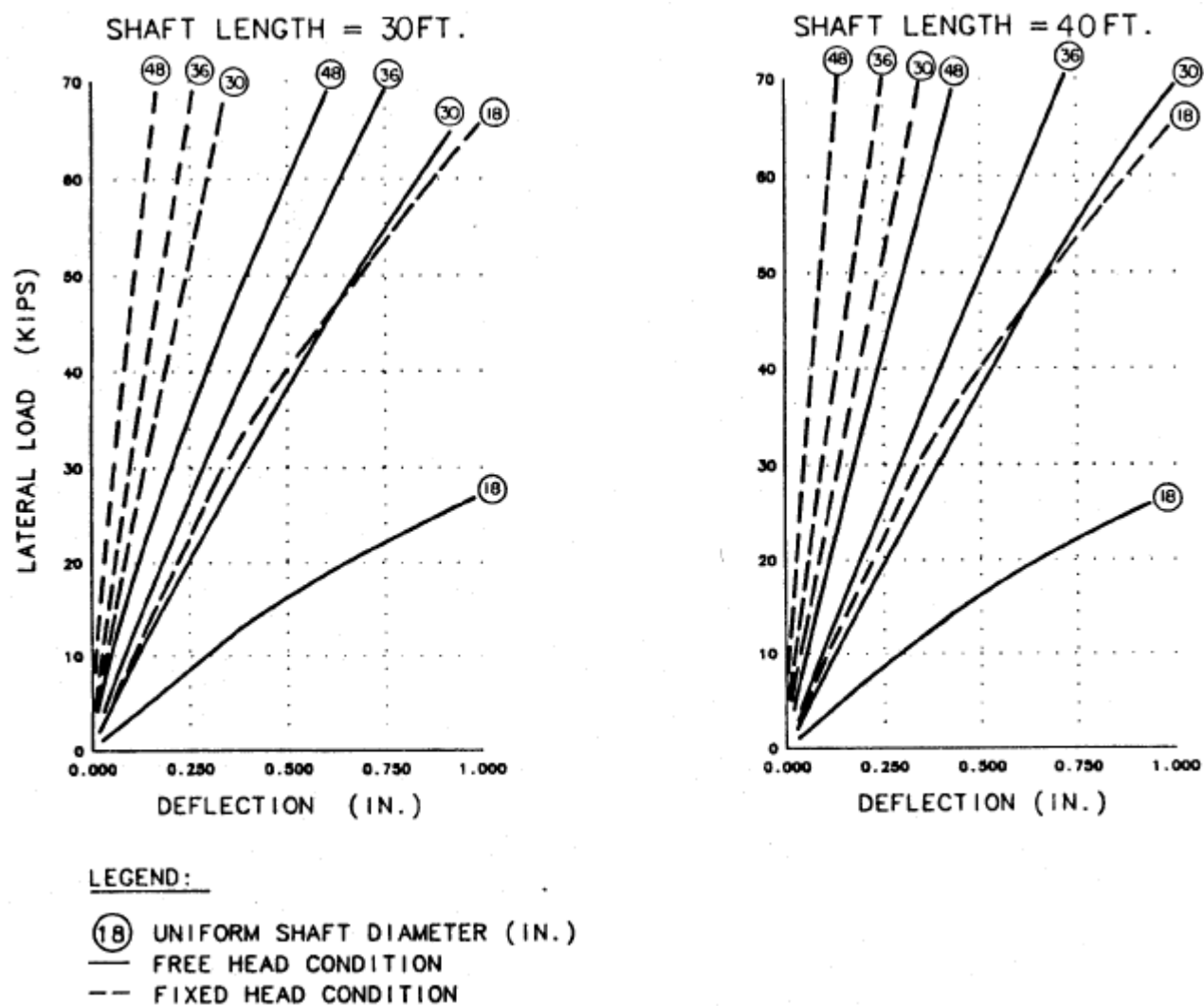


Figure 2.2-6B - COM624 Analyses for Preliminary Design
Lateral Load Versus Lateral Deflection, Static Loading, Subsurface Profile 5 (Continued)

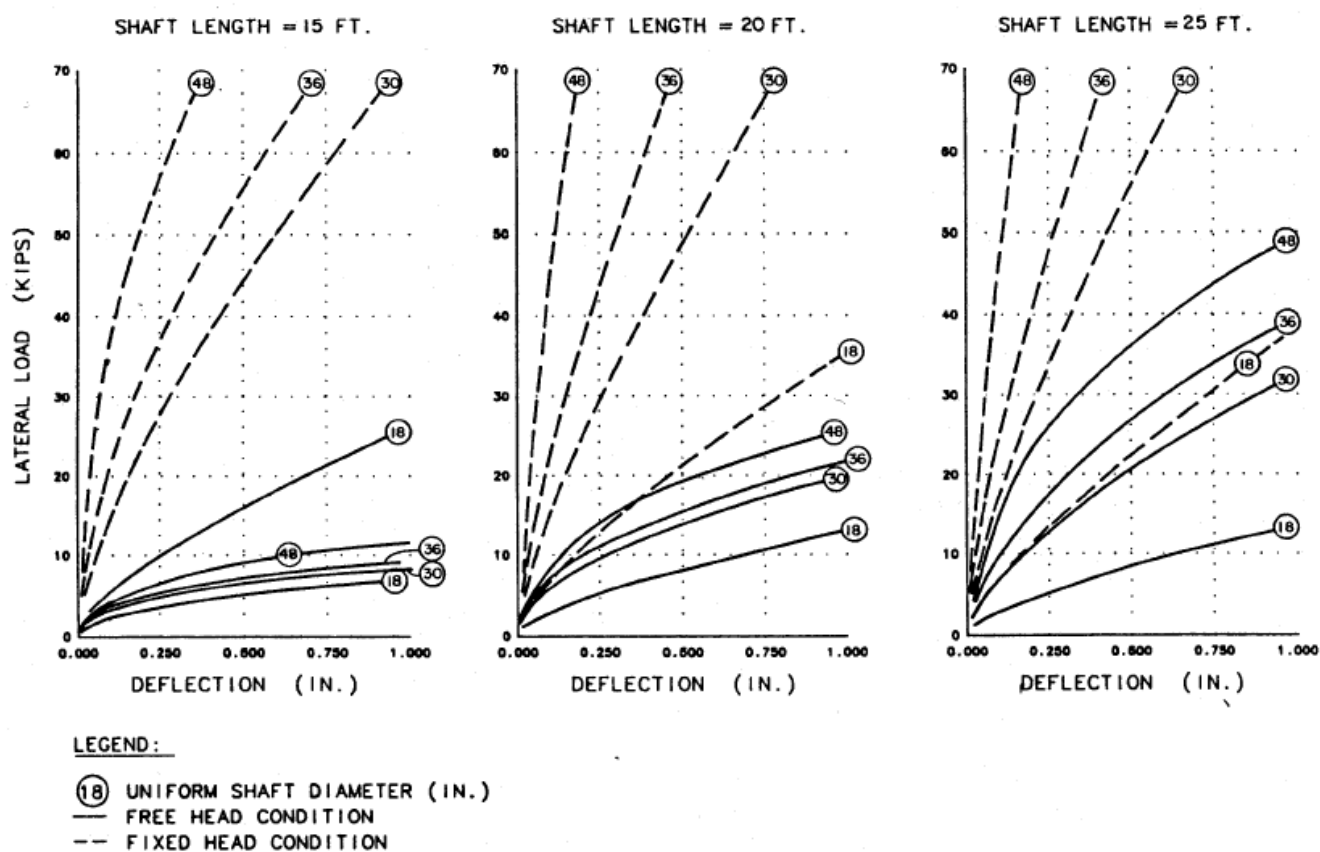


Figure 2.2-7A - COM624 Analyses for Preliminary Design.
Lateral Load versus Lateral Deflection, Static Loading, Subsurface Profile 6

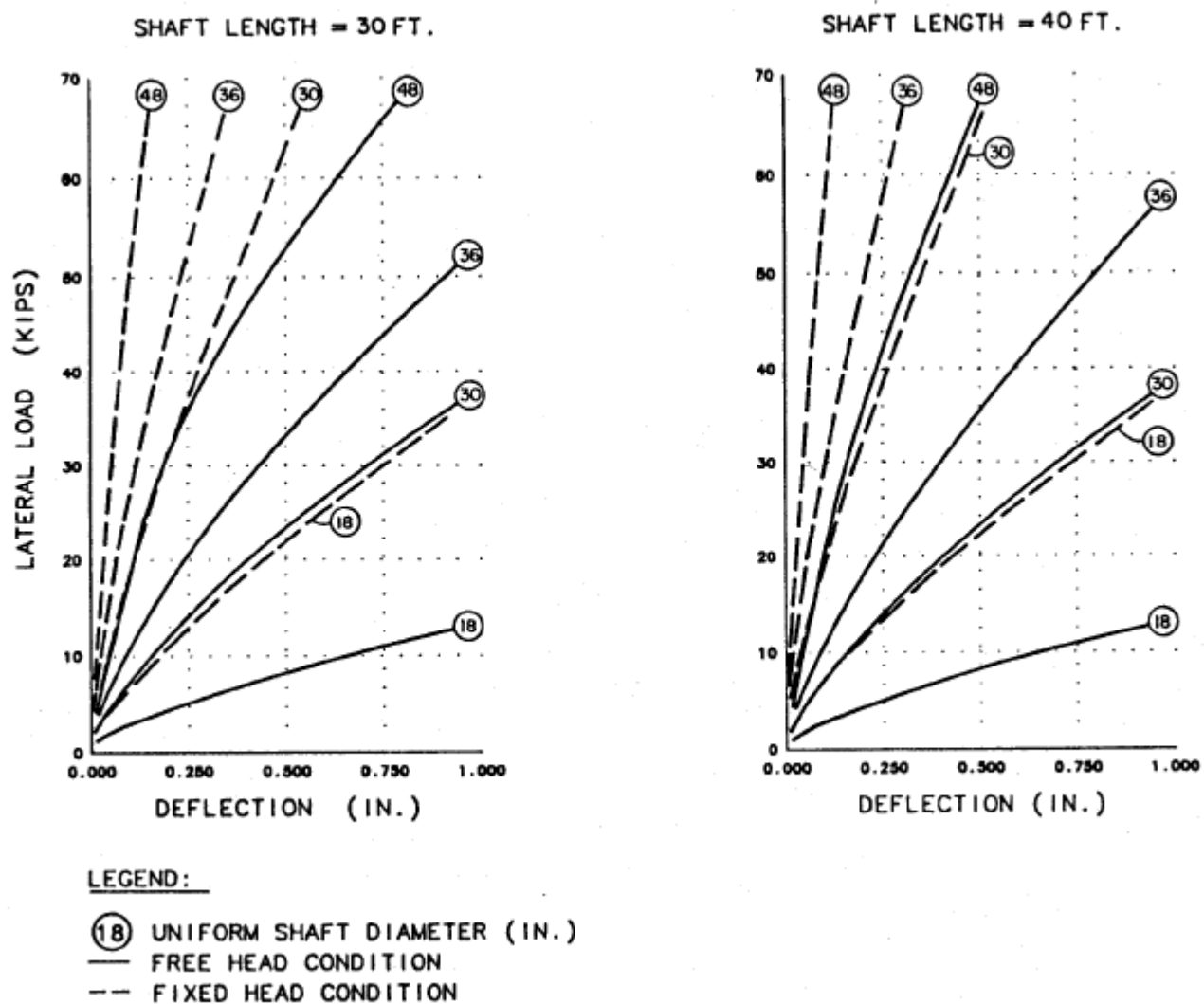


Figure 2.2-7B - COM624 Analyses for Preliminary Design.
Lateral Load versus Lateral Deflection, Static Loading, Subsurface Profile 6 (Continued)

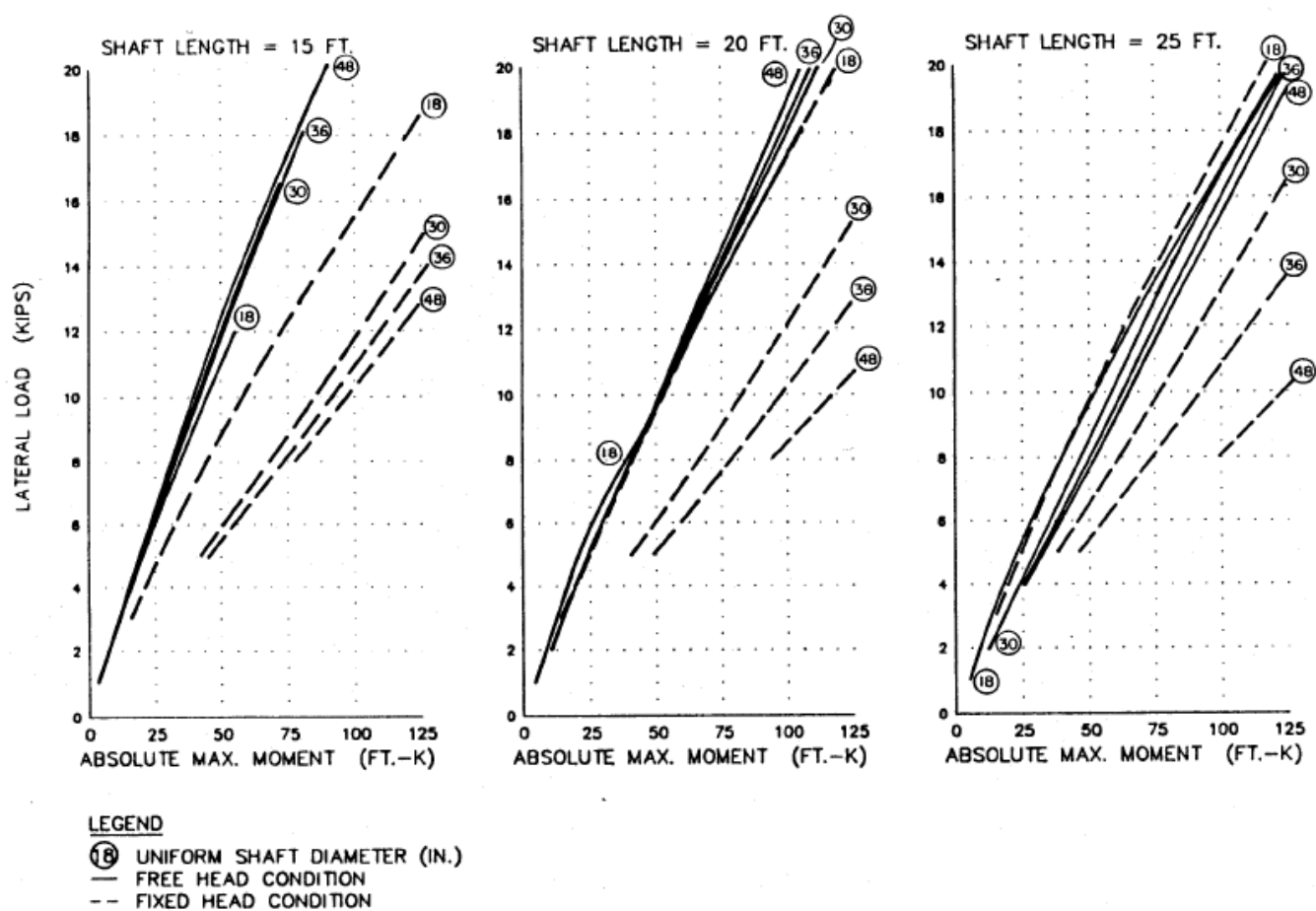


Figure 2.2-8A - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 1

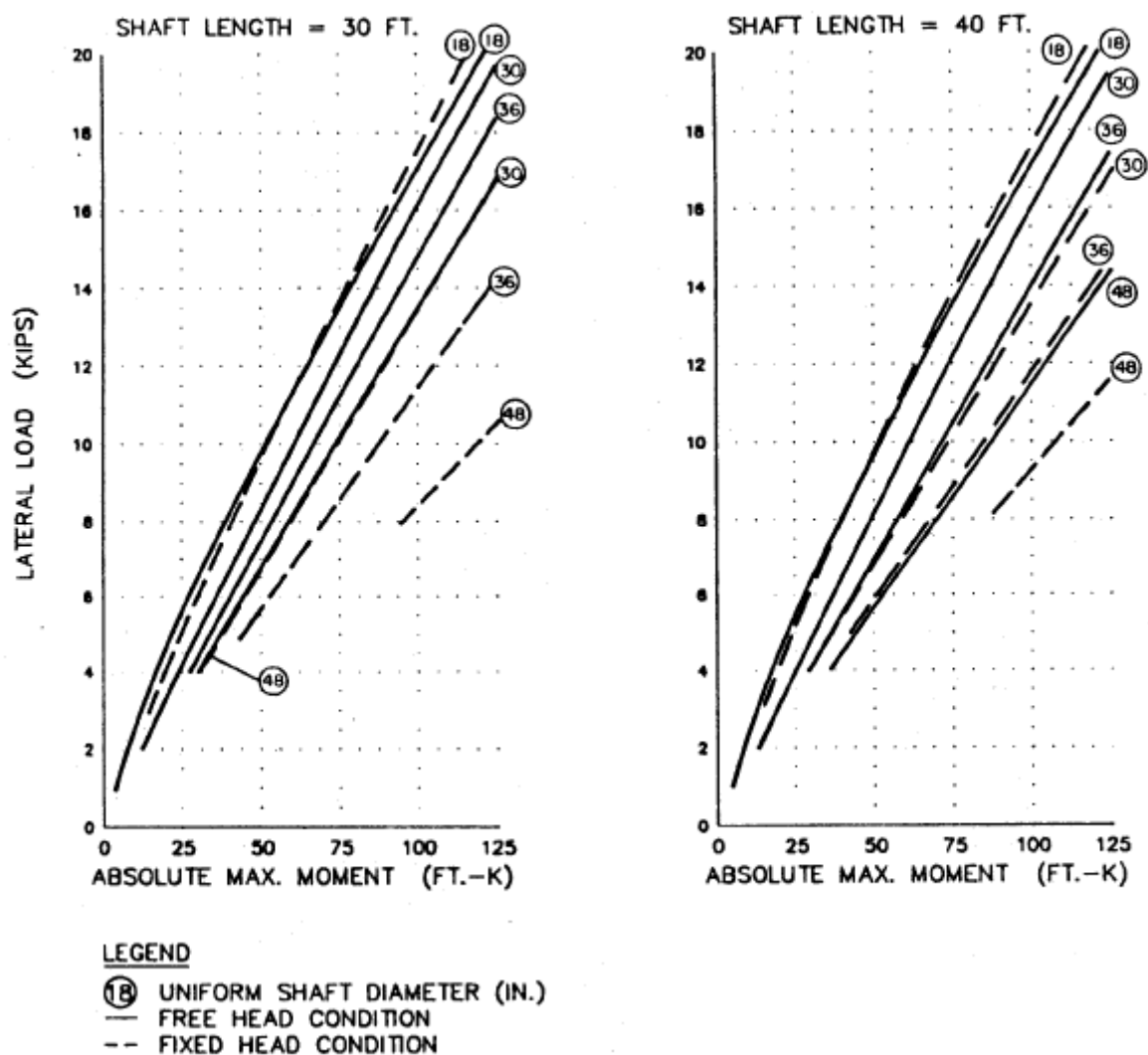


Figure 2.2-8B - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 1 (Continued)

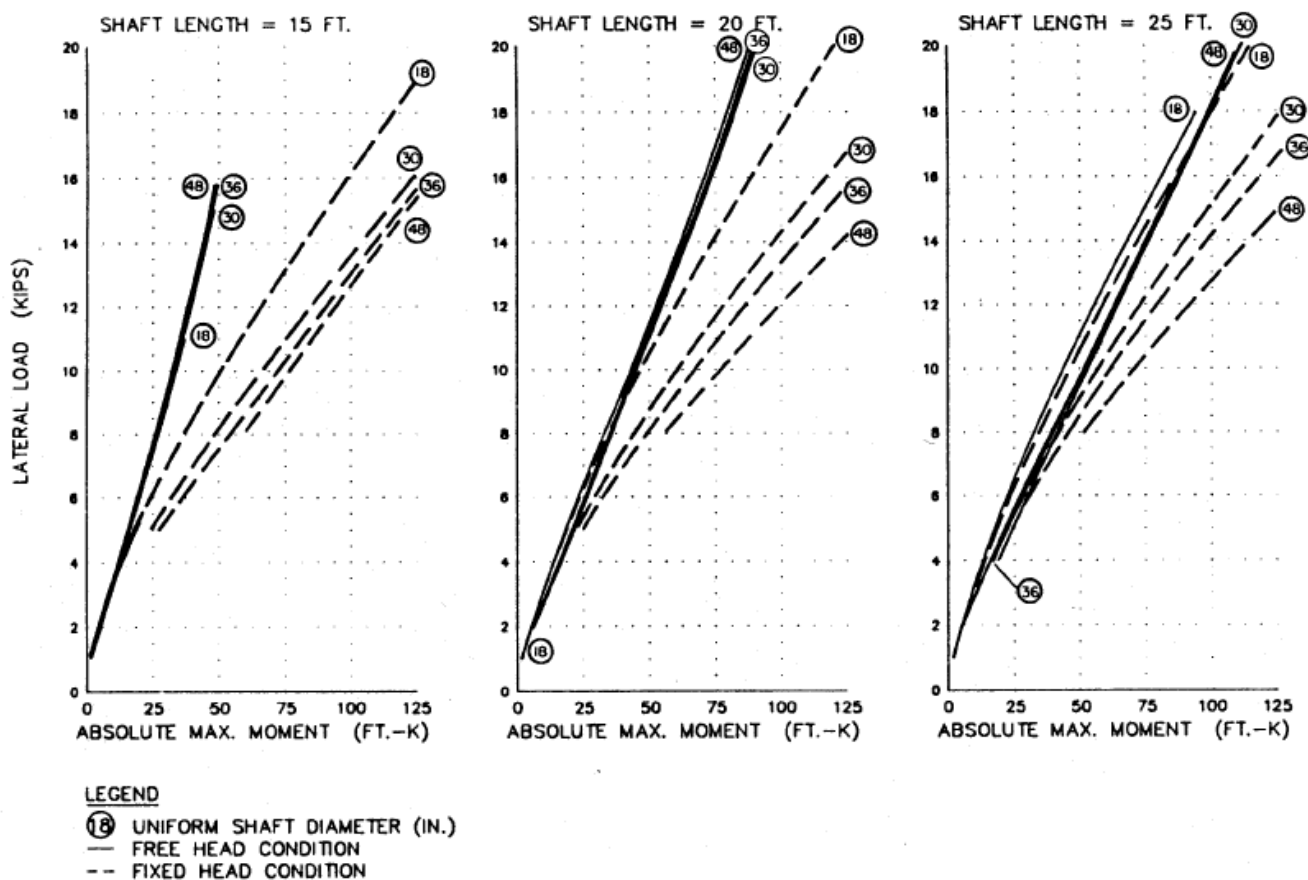


Figure 2.2-9A - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 3

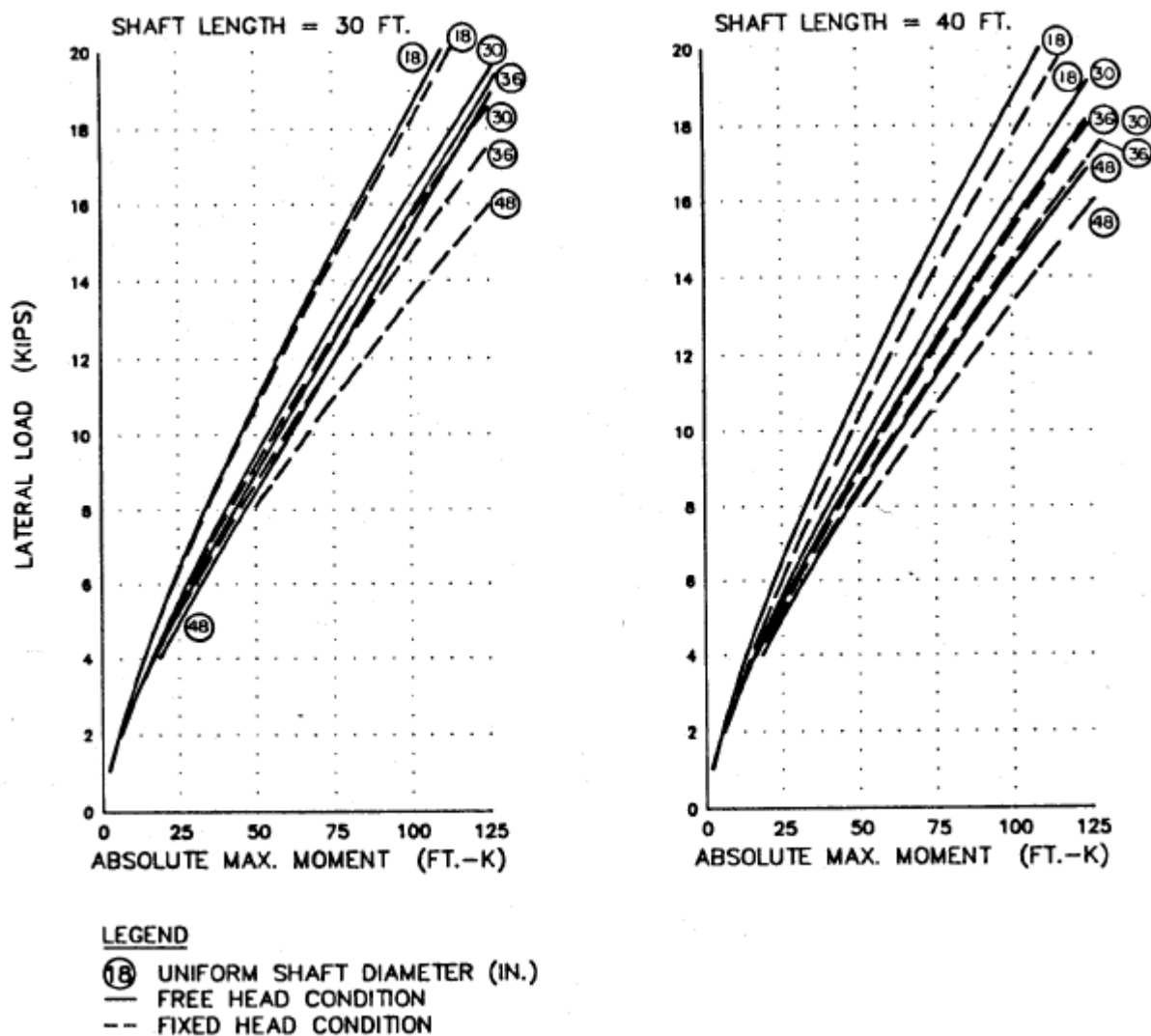


Figure 2.2-9B - COM624 Analyses for Preliminary Design
Lateral Load versus Absolute Maximum Moment, Static Loading, Subsurface Profile 3 (Continued)

Table 2.2-1 - Properties of Drilled Shafts used for COM624 Analyses

Nominal Diameter B (in.)	Cross-Sectional Area A (in. ²)	Gross Moment of Inertia I _c (in. ⁴)	Stiffness EcIc (lb-in. ²)
18	254	5,153	2.08x10 ¹⁰
30	707	39,761	16.0x10 ¹⁰
36	1,018	82,448	33.2x10 ¹⁰
48	1,810	260,576	105.0x10 ¹⁰

NOTES:

1. Shaft lengths: D = 15, 20, 25, 30 and 40 ft.
2. 28-day compressive strength of concrete: $f'_c = 5000$ psi
3. Modulus of elasticity of concrete: $E_c = 57,000 \sqrt{f'_c} = 4.03 \times 10^6$ psi
4. No reinforcement included

Table 2.2-2 - Comments on Input for Computer Program COM624

Item	Evaluation	Comments
Number of Shaft Increments	Determined as: $D/B + 15$	The effects of the shaft increment were examined and the expression was found to yield values leading to accurate solutions for the ground conditions modeled.
p-y Curves	Generated internally using: • Matlock [17] criteria for soft clay • Reese, et al.[4] criteria for stiff clay below the water table • Reese and Welch [5] criteria for stiff clay above the water table • Reese, et al, [6] criteria for sand	The user should be familiar with the methods of developing the load-deformation response for different soils presented in Section 2.3.1.1.1 noting the limiting assumptions.
Location of Butt of Shaft with Respect to Ground Surface	Assumed	Butt of the shaft was assumed level with the ground surface for all cases.
Soil properties: unit weight, angle of internal friction, undrained shear strength, strain at 50% stress level constant n_h in $E_s = n_h z$	Based on assumed soil type	Combinations of soil properties for the assumed soil types (i.e., loose sand, stiff clay) were chosen on the basis of consistency between properties and representativeness of soil type.
Lateral Loads	Variable incremental increase, at least to a level inducing a ground line deflection greater than 0.5 in.	Lateral loads were concentrated at the butt of the shaft. No distributed lateral loads were incorporated.

2.3 STRUCTURAL DESIGN AND GENERAL SHAFT DIMENSIONS

2.3.1 General

2.3.1.1 Final Design

2.3.1.1.1 COM624 Design Procedure

The computer program COM624 incorporates nonlinear load-deformation (p-y) curves to model the soil-foundation interaction, and adopts a finite-difference scheme to solve the problems. COM624 is capable of generating p-y curves internally using an input subsurface description with corresponding soil/rock properties, or can accept predetermined load-deformation curves for the soil/rock units. The method yields p-y data for the subsurface profile at specified increments along the shaft, if requested, and also computes the lateral deflection, rotation, moment, and shear at desired points along the foundation for a given loading. Recommendations by Reese [5] for constructing p-y curves for sand and clay are presented in 2.3.1.1.1.1 through 2.3.1.1.1.5. The recommended procedures for developing the p-y curves are derived from experimental studies, so inherent limitations exist where empirical coefficients are recommended based on particular lateral load tests.

The design of laterally loaded drilled shafts based on the deformation characteristics of the surrounding medium requires development of p-y curves for the soil and/or rock. The initial step in formulating nondimensional and computer solutions is the construction of representative p-y curves at various depths along the embedment of the shaft.

Theoretical soil mechanics and engineering experience provide the basis for developing p-y curves. Actual methods of constructing such curves arise from duplication of load-deformation behavior observed during lateral load tests. The methods are typically generalized through correlations with soil properties, shaft diameter, and depth, for broader application.

Because it is difficult to determine an exact percent of fixity and because increased shaft head fixity results in decreased ground line deflections, percent fixities somewhat less than those expected to be achieved are applied in deflection analysis of laterally-loaded drilled shafts so that the results obtained are somewhat conservative. It should be noted that the moment conditions at the two extremes of head fixity present the worst cases for consideration of the structural requirements of the shaft.

2.3.1.1.1.1 p-y Curves for Soft Clay Extending to the Ground Surface

(a) The following procedure can be used for developing p-y curves for short-term static loading as illustrated in Figure 2.3.1.1.1.1-1 [5].

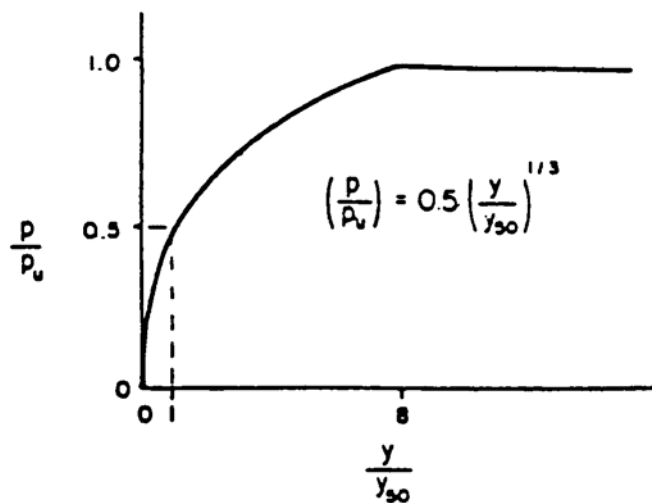
- (1) Obtain the best possible estimate of the variation with depth of undrained shear strength, c , and submerged unit weight, γ' . Also obtain the values of the strain, ϵ_{50} , corresponding to one-half the maximum principal-stress difference. If no stress-strain curves are available, assume a value of ϵ_{50} equal to 0.020, 0.010, and 0.005 for soft, medium, and stiff clay, respectively.
- (2) Compute the ultimate soil resistance per unit length of shaft, using the smaller of the values given by the equations below.

$$P_u = \left[3 + \frac{\gamma'}{c} z + \frac{J}{B} z \right] cB$$

$$P_u = 9 c B$$

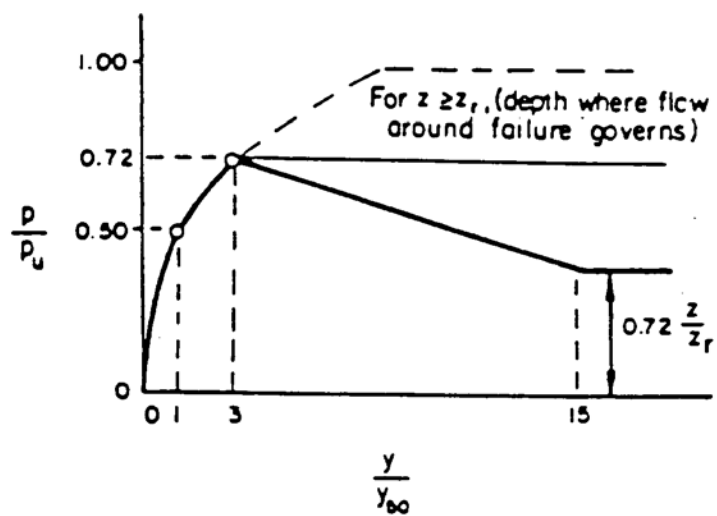
where:

- z = Depth from the ground surface to the p-y curve
- γ' = Average effective unit weight from the ground surface to depth, z
- c = Undrained shear strength at depth z
- B = Width of the shaft
- J = Empirical dimensionless parameter



Source: Reference 3

Figure 2.3.1.1.1.1-1 - Characteristic Shape of p - y Curves for Static Loading of Soft Clay below Water Table



Source: Adapted from Reference 19

Figure 2.3.1.1.1.1-2 Characteristic Shape of p - y Curves for Cyclic Loading of Soft Clay below Water Table

Matlock [17] states that the value of J was determined experimentally to be 0.5 for a soft clay and about 0.25 for a medium clay. A value of 0.5 is frequently used for J . The value of p_u is computed at each depth where a p - y curve is desired, based on shear strength at that depth. COM624 develops values of y and the corresponding p values at close spacings; if hand computations are being used, p - y curves shall be computed at depths to reflect the soil profile. If the soil is homogeneous, the p - y curves shall be obtained at close spacings near the ground surface where the shaft deflection is greater.

- (3) Compute the deflection, y_{50} , at one-half the ultimate soil resistance from the following equation:

$$y_{50} = 2.5 \epsilon_{50} B$$

- (4) Points describing the p - y curve are computed from the following relationship:

$$\frac{p}{p_u} = 0.5 \left(\frac{y}{y_{50}} \right)^{1/3}$$

The value of p remains constant beyond $y = 8 y_{50}$.

- (b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.1-2.

- (1) Construct the p - y curve in the same manner as for short-term static loading for values of p less than $0.72 p_u$.
- (2) Solve the first two equations in 2.3.1.1.1 simultaneously to find the depth, z_r , where the transition occurs from a wedge-type failure to a flow-around failure. If the unit weight and shear strength are constant in the upper zone, then

$$z_r = \frac{6 c B}{(\gamma' B + J_c)}$$

If the unit weight and shear strength vary with depth, the value of z_r shall be computed with the soil properties at the depth where the p - y curve is desired.

- (3) If the depth to the p - y curve is greater than or equal to z_r , then p is equal to $0.72 p_u$ for all values of y greater than $3 y_{50}$.
- (4) If the depth to the p - y curve is less than z_r , then the value of p decreases from $0.72 p_u$ at $y = 3 y_{50}$ to the value given by the following expression at $y = 15 y_{50}$.

$$p = 0.72 p_u \left(\frac{z}{z_r} \right)$$

The value of p remains constant beyond $y = 15 y_{50}$.

- (c) Recommended Soil Tests

For determining the various shear strengths of the soil required in the p - y construction, Matlock [17] recommended the following tests in order of preference:

- (1) In situ vane-shear tests with parallel sampling for soil identification
- (2) Unconsolidated-undrained triaxial compression tests having a confining stress equal to the overburden pressure, with c being defined as half the total maximum principal stress difference
- (3) Miniature vane tests of samples in tubes

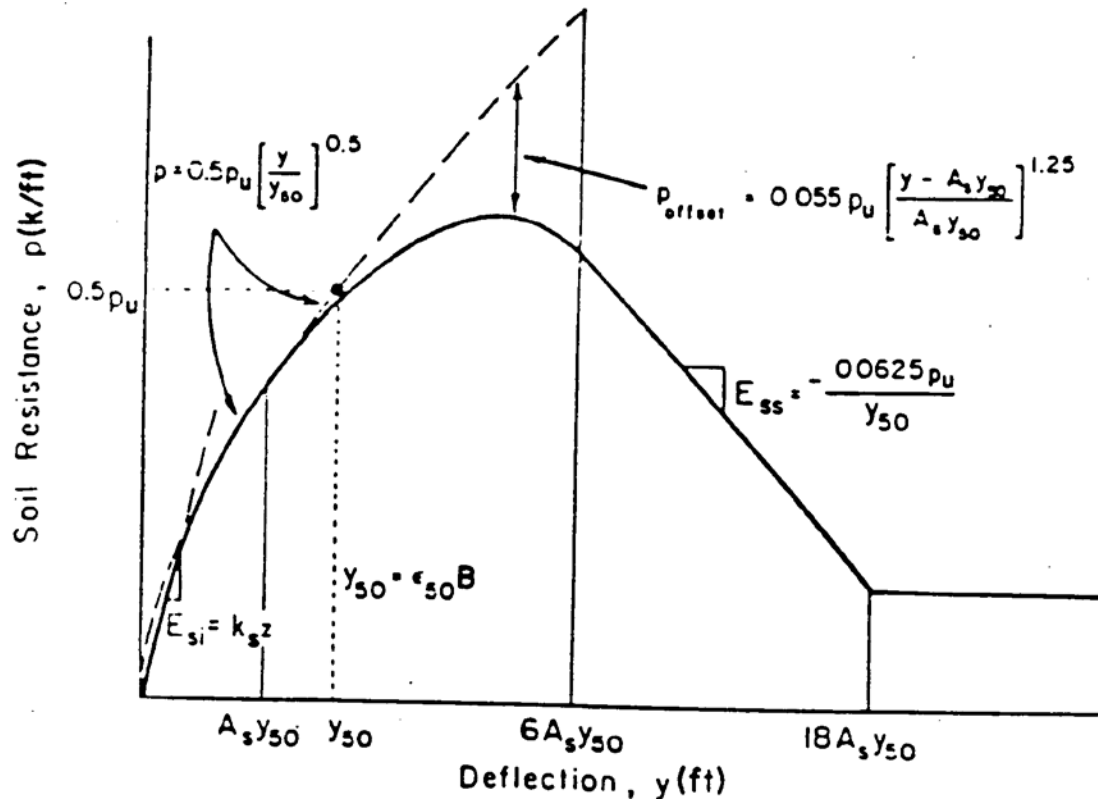
(4) Unconfined compression tests

Matlock's recommendations are related to the determination of the shear strength of the clay. If field vane tests are performed, for example, other tests shall be performed to obtain ϵ_{50} and the unit weight for the soil.

In the test program, emphasis shall be placed on determining soil properties at or near the ground surface. In general, the farther the soil from the ground surface, the less is its influence on shaft response. For a soil profile that is relatively homogeneous, studies have shown that the soil below 10 shaft diameters has only a small influence on shaft deflection and bending moment.

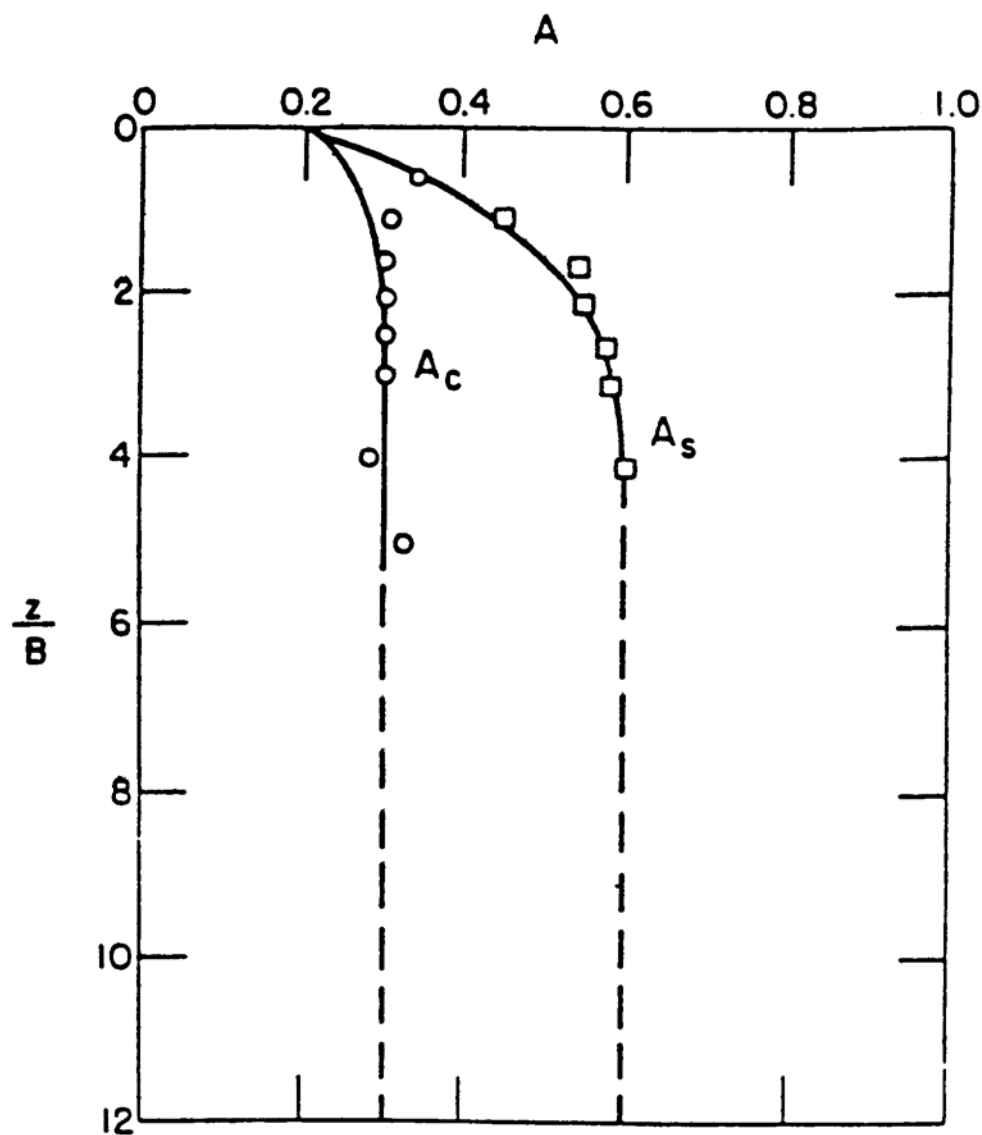
2.3.1.1.1.2 *p-y Curves for Stiff Clay Below the Water Table*

- (a) The following procedure can be used for developing p-y curves for short-term static loading as illustrated in Figure 2.3.1.1.2-1. The empirical parameters, A_s and A_c , shown in Figure 2.3.1.1.2-2, and values of k_s and k_c are based on the results of experiments. Values of k_s for static loading shall be assumed to equal 425, 850 and 1,700 kips per cubic foot (kef) for soft, medium, and stiff clay soils, respectively. Values of k_c for cyclic loading shall be assumed to equal 170, 350 and 700 kef for soft, medium, and stiff clay soils, respectively.



Source: Adapted from Reference 18

Figure 2.3.1.1.2-1 - Characteristic Shape of p-y Curves for Static Loading in Stiff Clay below Water Table



Source: Adapted from Reference 5

Figure 2.3.1.1.2-2 - Values of Constants A_s and A_c

- (1) Obtain the values of the undrained soil shear strength, c , soil submerged unit weight, γ' , and shaft diameter, B .
- (2) Compute the average undrained soil shear strength, c_a , over the depth, z .
- (3) Compute the ultimate soil resistance per unit length of shaft using the smaller of the values given by the equations below:

$$p_u = 2 c_a B + \gamma' B z + 2.83 c_a z$$

$$p_u = 11 c B$$

- (4) Choose the appropriate value of A_s from Figure 2.3.1.1.1.2-2 for the particular nondimensional depth.
- (5) Establish the initial straight-line portion of the p - y curve:

$$p = (kz) y$$

Use the appropriate value of k_s or k_c for k .

- (6) Compute the following:

$$y_{50} = \epsilon_{50} B$$

Use an appropriate value of ϵ_{50} from results of laboratory test or, in the absence of laboratory tests, assume a value of ϵ_{50} equal to 0.020, 0.010, or 0.005 for soft, medium, and stiff clay, respectively.

- (7) Establish the first parabolic portion of the p - y curve using the following equation and obtaining p_u from the equations in (3) above.

$$p = 0.5 p_u \sqrt{\frac{y}{y_{50}}}$$

This equation defines the portion of the p - y curve from the point of the intersection with the equation in (5) above to a point where y is equal to $A_s y_{50}$ (see note in step (10) below).

- (8) Establish the second portion of the parabolic p - y curve using the following relationship:

$$p = 0.5 p_u \left[\frac{y}{y_{50}} \right]^{0.5} - 0.55 p_u \left[\frac{y - A_s y_{50}}{A_s y_{50}} \right]^{1.25}$$

This equation defines the portion of the p - y curve from the point where y is equal to $A_s y_{50}$ to a point where y is equal to $6 A_s y_{50}$ (see note in step (10) below).

- (9) Establish the next straight-line portion of the p - y curve.

$$p = 0.5 p_u \sqrt{6 A_s} - 0.411 p_u \left(\frac{0.0625}{y_{50}} \right) p_u (y - 6 A_s y_{50})$$

This equation defines the portion of the p - y curve from the point where y is equal to $6 A_s y_{50}$ to a point where y is equal to $18 A_s y_{50}$ (see note in step (10) below).

- (10) Establish the final straight-line portion of the p-y curve.

$$p = 0.5 p_u \sqrt{6 A_s} - 0.411 p_u - 0.75 p_u A_s$$

or

$$p = p_u (1.225 \sqrt{A_s} - 0.75 A_s - 0.411)$$

The above equation defines the portion of the p-y curve from the point where y is equal to $18 A_s y_{50}$ and for all larger values of y (see following note).

Note: The step-by-step procedure is outlined and Figure 2.3.1.1.2-1 is drawn as if there is an intersection between the equations in (5) and (7) above. However, there may be no intersection of the former with any of the other equations defining the p-y curve. If there is no intersection, the equation shall be employed that gives the smallest value of p for any value of y.

- (b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.2-3. Steps (1), (2), (3), (5), and (6) are the same as for the static case.

- (4) Choose the appropriate value of A_c from Figure 2.3.1.1.2-2 for the particular nondimensional depth.

Compute the following:

$$y_p = 4.1 A_s y_{50}$$

- (7) Establish the parabolic portion of the p-y curve.

$$p = A_c P_u \left(1 - \left(\frac{y - 0.45 y_p}{0.45 y_p} \right)^{2.5} \right)$$

This equation defines the portion of the p-y curve from the point of the intersection with the equation in step (5) to the point where y is equal to $0.6 y_p$ (see note in step (9) below).

- (8) Establish the next straight-line portion of the p-y curve.

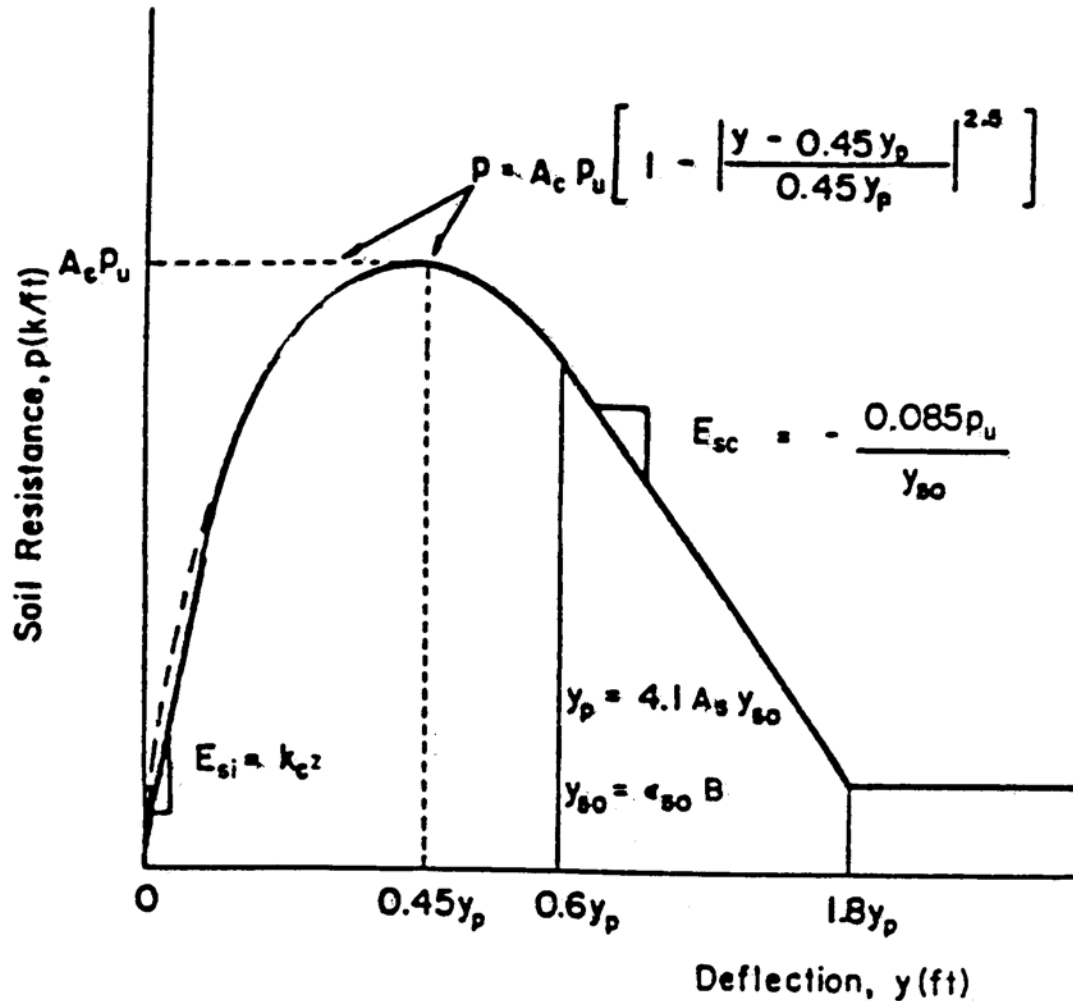
$$p = 0.936 A_c p_u - \frac{0.085}{y_{50}} p_u (y - 0.6 y_p)$$

This equation defines the portion of the p-y curve from the point where y is equal to $0.6 y_p$ to the point where y is equal to $1.8 y_p$ (see note in step (9) below).

- (9) Establish the final straight-line portion of the p-y curve.

$$p = 0.936 A_c p_u - \frac{0.102}{y_{50}} p_u y_p$$

This equation shall define the portion of the p-y curve from the point where y is equal to $1.8 y_p$ and for all larger values of y (see following note).



Source: Adapted from Reference 18

Figure 2.3.1.1.2-3 - Characteristic Shape of p - y Curves for Cyclic Loading in Stiff Clay below Water Table

Note: The step-by-step procedure is outlined and Figure 2.3.1.1.2-3 is drawn as if there is an intersection between the equations in steps (5) and (7) above. However, there may be no intersection of these two equations, and there may be no intersection of the former with any of the other equations defining the p - y curve. If there is no intersection, the equation shall be employed that gives the smallest value of p for any value of y .

(c) Recommended Soil Tests

Triaxial compression tests of the unconsolidated-undrained type with confining pressures conforming to the in situ total overburden pressures are recommended for determining the shear strength of the soil. The value of ϵ_{80} shall be taken as the strain during the test corresponding to the stress equal to half the maximum total-principal-stress difference. The shear strength, c , shall be interpreted as one-half of the maximum total-stress difference. Values obtained from the triaxial tests might be somewhat conservative but would represent more realistic strength values than other tests. The unit weight of the soil shall also be determined.

2.3.1.1.1.3 *p-y Curves for Stiff Clay Above the Water Table*

(a) The following procedure can be used for developing *p-y* curves for short-term static loading as illustrated in Figure 2.3.1.1.1.3-1.

- (1) Obtain the values of the undrained shear strength, c , soil unit weight, γ , and shaft diameter, B . Also obtain the values of ϵ_{50} from stress-strain curves. If stress-strain curves are not available, use a value from ϵ_{50} of 0.010 or 0.005 for medium or stiff clay with the larger value being more conservative.
- (2) Compute the ultimate soil resistance per unit length of shaft, p_u , using the smaller of the values given by the equations below (see 2.3.1.1.1).

$$p_u = \left[3 + \frac{\gamma'}{c} Z + \frac{J}{B} Z \right] cB$$

$$p_u = 9c B$$

In the use of the first equation above, the shear strength shall be taken as the average from the ground surface to the depth being considered, and J is taken as 0.5. The unit weight of the soil shall reflect the position of the water table.

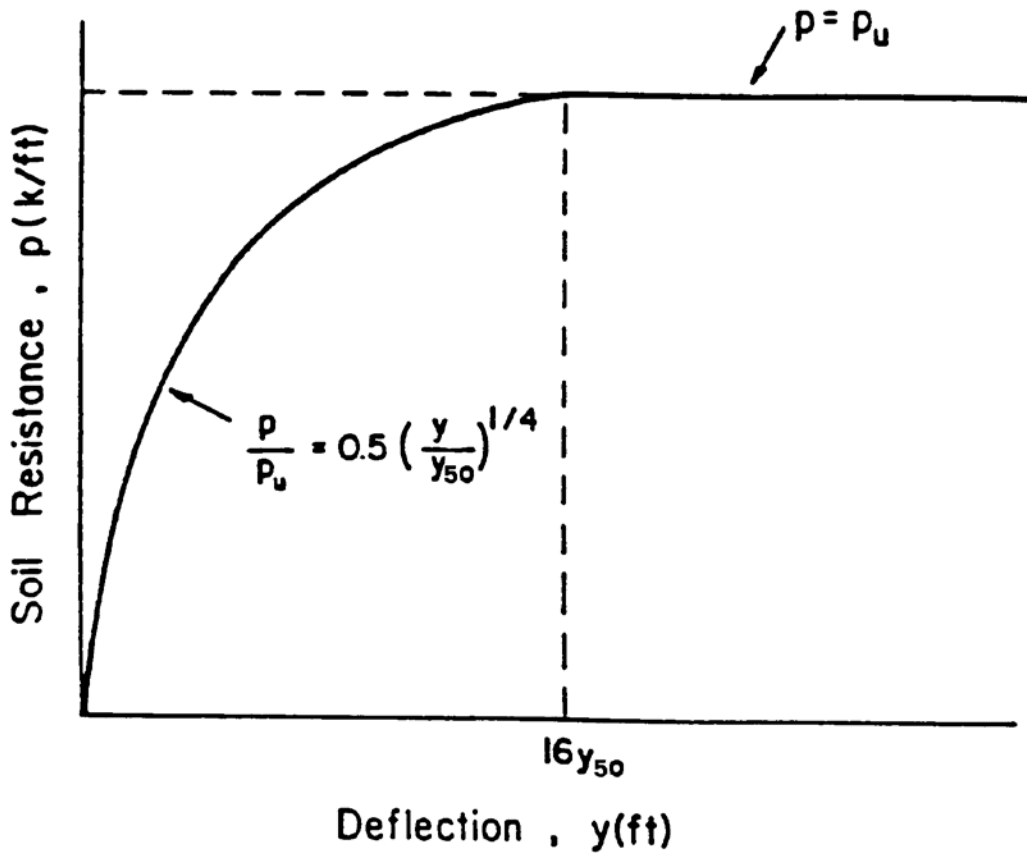
- (3) Compute the deflection, y_{50} , at one-half the ultimate soil resistance from the following equation:

$$y_{50} = 2.5 \epsilon_{50} B$$

- (4) Points describing the *p-y* curve may be computed from the relationship below.

$$\frac{p}{p_u} = 0.5 \left(\frac{y}{y_{50}} \right)^{1/4}$$

- (5) Beyond $y = 16 y_{50}$, p is equal to p_u for all values of y .



Source: Adapted from Reference 5

Figure 2.3.1.1.3-1 Characteristic Shape of p-y Curves for Static Loading in Stiff Clay above Water Table

(b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.3-2.

- (1) Determine the p-y curve for short-term static loading by the procedure previously given.
- (2) Determine the number of times the design lateral load will be applied to the shaft.
- (3) For several values of p / p_u , obtain the value of the parameter describing the effect of repeated loading on deformation, C , from a relationship developed by laboratory tests, or, in the absence of tests, from the following equation:

$$C = 9.6 \left(\frac{p}{p_u} \right)^4$$

- (4) At the value of p corresponding to the values of p / p_u selected in step (3), compute new values of y for cyclic loading from the following equation:

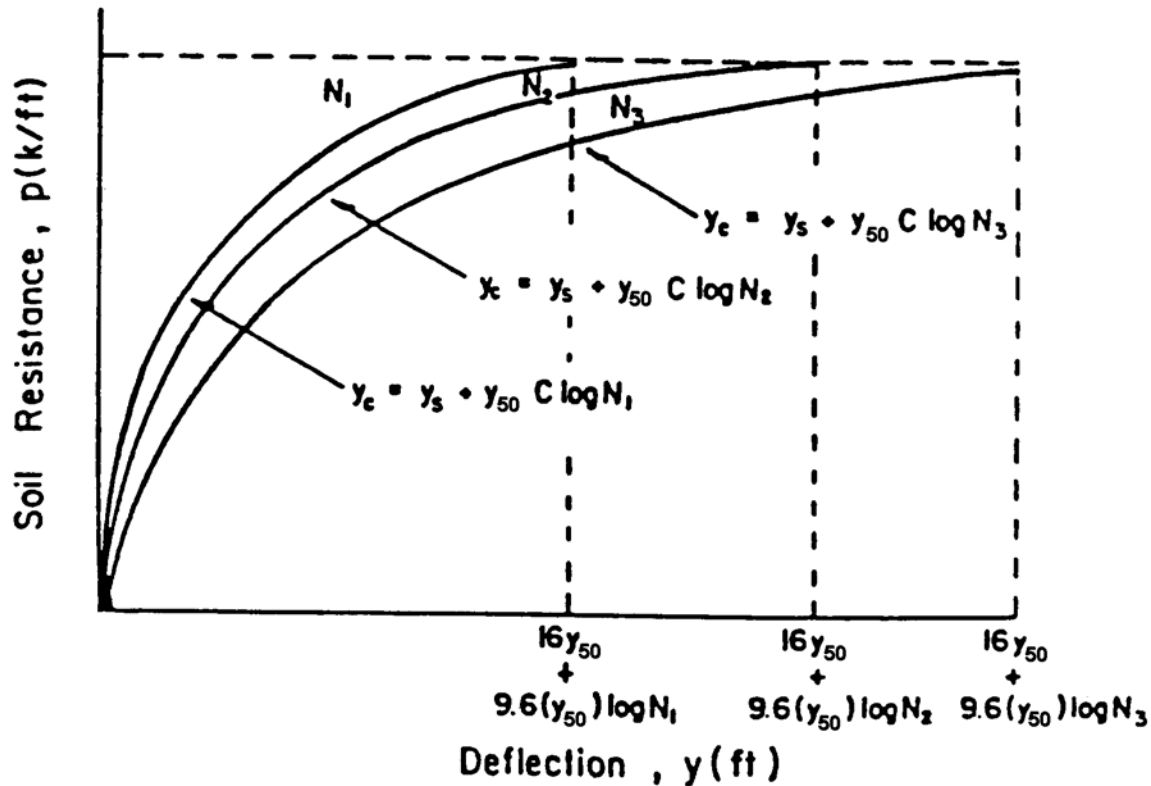
$$y_c = y_s + y_{50} C \log N$$

where y_s is the deflection under the short-term static load, p .

- (5) The p-y curve defines the soil response after N cycles of load.

(c) Recommended Soil Tests

Triaxial compression tests of the unconsolidated-undrained type with confining stresses equal to the overburden pressures at the elevations from which the samples were taken are recommended to determine the shear strength. The value of ϵ_{50} shall be taken as the strain during the test corresponding to the stress equal to one-half the maximum total principal stress difference. The undrained shear strength, c , shall be defined as one-half the maximum total principal stress difference. The unit weight of the soil shall also be determined.



Source: Adapted from Reference 5

Figure 2.3.1.1.3-2 Characteristic Shape of p - y Curves for Cyclic Loading in Stiff Clay above Water Table

2.3.1.1.4 p - y Curves for Clay Below Water Table by Unified Method

- (a) The following procedure can be used for developing p - y curves for short-term static loading as illustrated in Figure 2.3.1.1.4-1.
 - (1) Obtain the values of the undrained shear strength, c , the submerged unit weight, γ' , and the shaft diameter, B . Also obtain values of ϵ_{50} from stress-strain curves. If stress-strain curves are not available, assume a value of ϵ_{50} equal to 0.020 for very soft clay, 0.010 for soft clay, 0.007 for medium clay, 0.005 for stiff clay, and 0.004 for very stiff clay.

- (2) Compute c_a and σ'_v , for $z < 12 B$, where c_a equals the average undrained shear strength, σ'_v equals average effective stress, and z is the depth.
- (3) Compute the variation of p_u with depth using the equations below.

For $z < 12 B$, p_u is the smaller of the values computed from the two equations below.

$$p_u = \left(2 + \frac{\sigma'_v}{c_a} + 0.833 \frac{z}{B} \right) c_a B$$

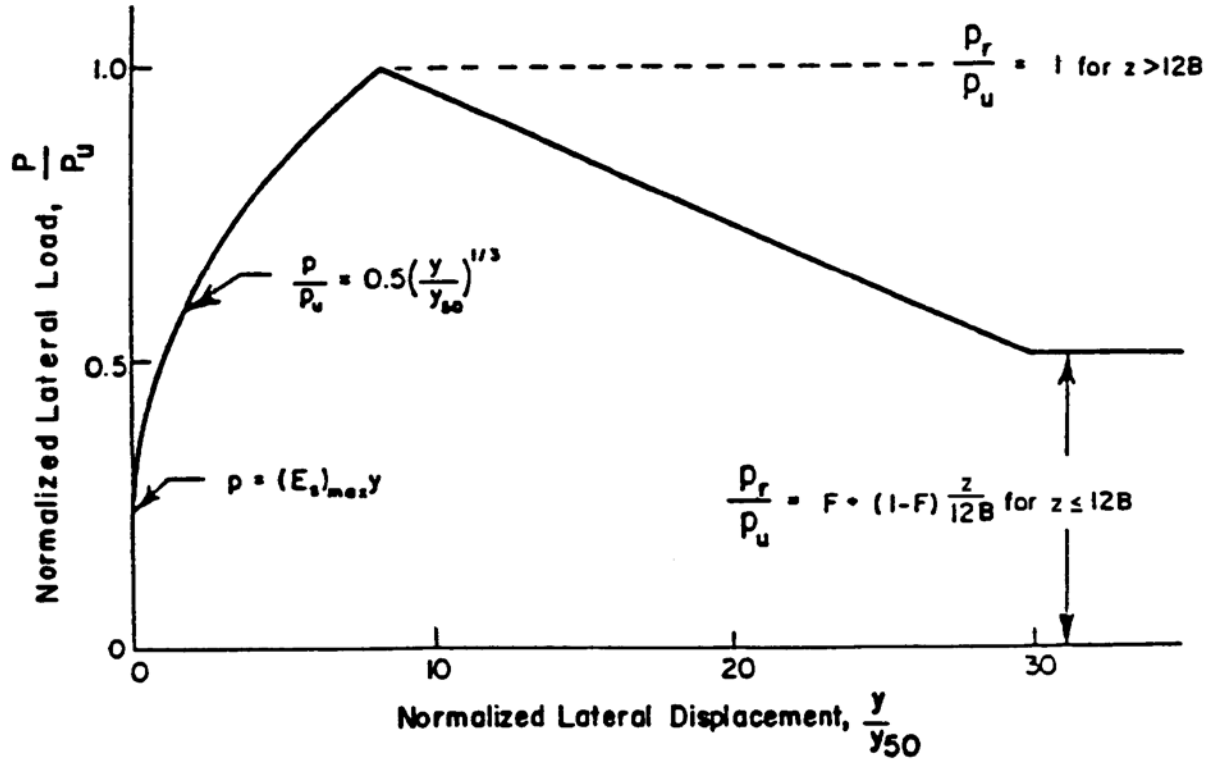
$$p_u = \left(3 + 0.5 \frac{z}{B} \right) c B$$

For $z > 12 B$

$$p_u = 9 c B$$

The steps below are for a particular depth, z .

- (4) Select the coefficients A and F as indicated below. The coefficients A and F are determined empirically based on the results of load tests. For normally consolidated inorganic clay, values of A and F are of the order of 2.5 and 1.0, respectively. For heavily overconsolidated inorganic clays, values of A and F are of the order of 0.35 and 0.5, respectively. The recommended procedure for estimating A and F for other clays is given below.
 - Determine as many of the following properties of the clay as possible, including c , ε_{50} , overconsolidation ratio, sensitivity, degree of fissuring, and Atterberg limits.
 - Compare the properties of the soil in question to the properties of typical normally and heavily overconsolidated, inorganic clays. If the properties are similar to either clay type, use A and F for the similar clay.
 - If the properties are not similar to either, estimate A and F using engineering judgment.



Source: Adapted from Reference 5

Figure 2.3.1.1.4-1 Characteristic Shape of p-y Curves for Unified Clay Criteria, Static Loading

- (5) Compute: $y_{50} = A \varepsilon_{50} B$
- (6) Determine $E_{s \max}$. When no other method is available, use the equation below.

$$E_{s \max} = n_h z$$

Assume values of n_h equal to 25 kips per cubic foot (kcf) for very soft clay, 85 kcf for soft clay, 250 kcf for medium clay, 850 kcf for stiff clay, and 2,500 kcf for very stiff clay.

- (7) Use the following equation to compute the deflection at the intersection between the initial linear portion and curved portion.

$$y_g = \left[\frac{0.5 p_u}{(E_s)_{\max}} \right]^{1.5} (y_{50})^{-0.5}$$

y_g can be no larger than $8 y_{50}$.

- (8) For $0 < y < y_g$,

$$p = E_{s \max} y$$

For $y_g < y < 8 y_{50}$

$$p = 0.5 p_u \left(\frac{y}{y_{50}} \right)^{1/3}$$

For $8y_{50} < y < 30y_{50}$

$$p = p_u + \frac{p_r - p_u}{22 y_{50}} (y - 8y_{50})$$

where:

$$p_r = p_u \left[F + (1 - F) \frac{z}{12B} \right]$$

p_r will be equal to or less than p_u .

For $y > 30 y_{50}$

$$p = p_r$$

(b) The following procedure is for cyclic loading and is illustrated in Figure 2.3.1.1.1.4-2.

(1) Repeat steps (1) through (8) for static loading.

(2) Compute:

$$p_{cr} = 0.5 p_u \left[\frac{z}{12B} \right] \leq 0.5 p_u$$

For $y_g < y < y_{50}$

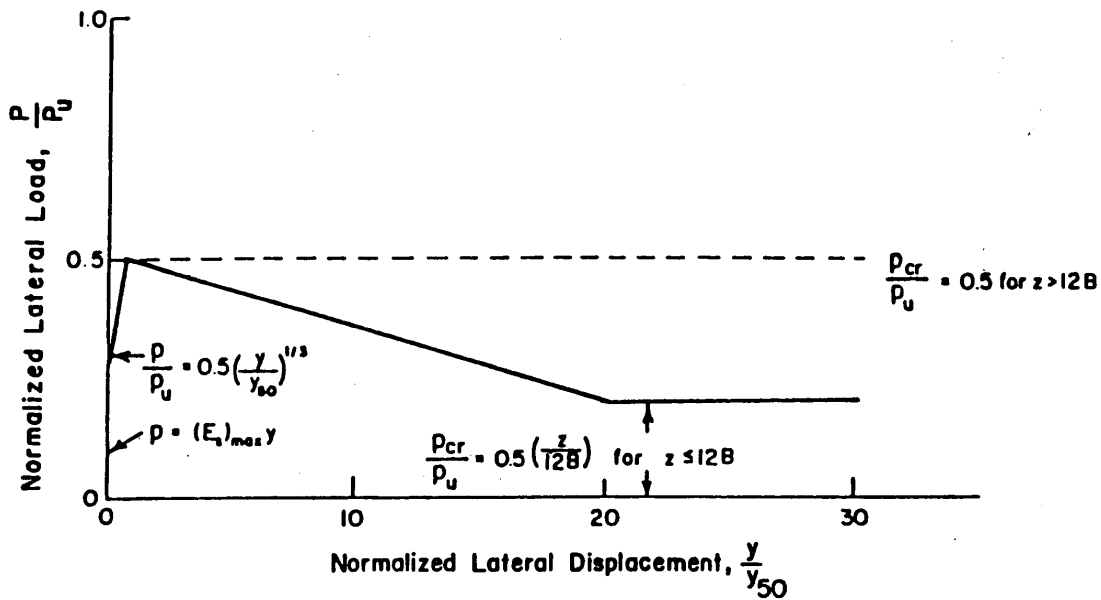
$$p = 0.5 p_u \left[\frac{y}{y_{50}} \right]^{1/3}$$

For $y_{50} < y < 20y_{50}$

$$p = 0.5 p_u + \left[\frac{p_{cr} - 0.5 p_u}{19 y_{50}} \right] (y - y_{50})$$

For $y > 20 y_{50}$

$$p = p_{cr}$$



Source: Adapted from Reference 5

Figure 2.3.1.1.4-2 - Characteristic Shape of p-y Curves for Unified Clay Criteria, Cyclic Loading

2.3.1.1.1.5 p-y Curves for Sand

- (a) The following procedure can be used for developing p-y curves for short-term static loading and for cyclic loading of sand as illustrated in Figure 2.3.1.1.5-1.

- (1) Obtain values for the angle of internal friction, ϕ' , the effective soil unit weight, γ' , and shaft diameter, B.
- (2) Make the following preliminary computations:

$$\alpha = \phi/2$$

$$\beta = 45 + \phi/2$$

$$K_o = 0.4$$

$$K_a = \tan^2 (45 - \phi/2)$$

$$K_p = \tan^2 (45 + \phi/2)$$

- (3) Compute the ultimate soil resistance per unit length of shaft using the smaller of the values given by the equations below, where z is equal to the depth below the ground surface.

$$p_{st} = \gamma B^2 [S_1 (z/B) + S_2 (z/B)^2]$$

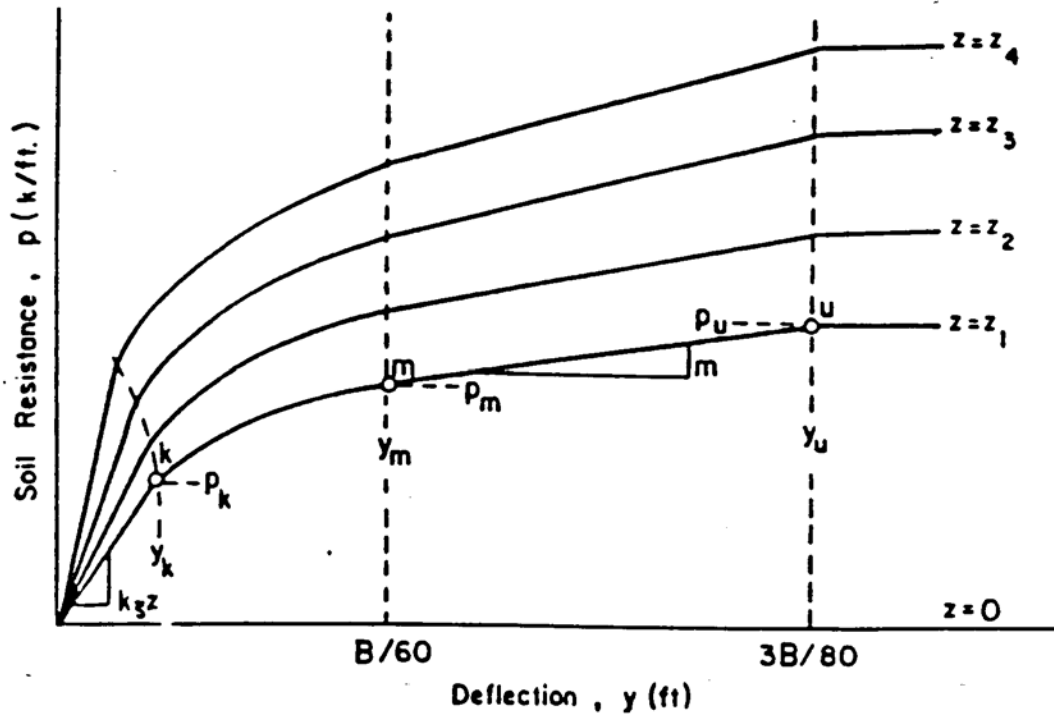
$$p_{sd} = \gamma B^2 [S_3 (z/B)]$$

where:

$$S_1 = (K_p - K_a)$$

$$S_2 = \tan \beta (K_p \tan \alpha + K_o [\tan \phi \sin \beta (\sec \alpha + 1) - \tan \alpha])$$

$$S_3 = K_p^2 (K_p + K_p \tan \phi) - K_a$$



Source: Adapted from Reference 5

Figure 2.3.1.1.5-1 - Characteristic Shape of a Family of p-y Curves for Static and Cyclic Loading in Sand

- (4) The depth of transition, z_t , can be found by equating the expressions in the first two equations in (3) above as follows:

$$z_t / B = (S_3 - S_1) / S_2$$

The appropriate γ for the position of the water table shall be employed. Above z_t , use the first equation in (3). Below z_t , use the second equation in (3). S_1 , S_2 , S_3 , and z_t/B are functions only of ϕ , therefore, the values shown in Table 2.3.1.1.5-1 can be computed.

- (5) Select a depth at which a p-y curve is desired.
- (6) Establish y_u as $3B/80$. Compute p_u using one of the following equations:

$$p_u = \bar{A}_s p_s$$

$$p_u = \bar{A}_c p_s$$

Use the appropriate value of $\bar{A}_s$ or $\bar{A}_c$ from Figure 2.3.1.1.5-2 for the particular nondimensional depth, and for either the static or cyclic case. Refer to the computation in Step (4) for the appropriate equation to use for p_s .

- (7) Establish y_m as $B/60$. Compute p_m using one of the following equations:

$$p_m = B_s p_s$$

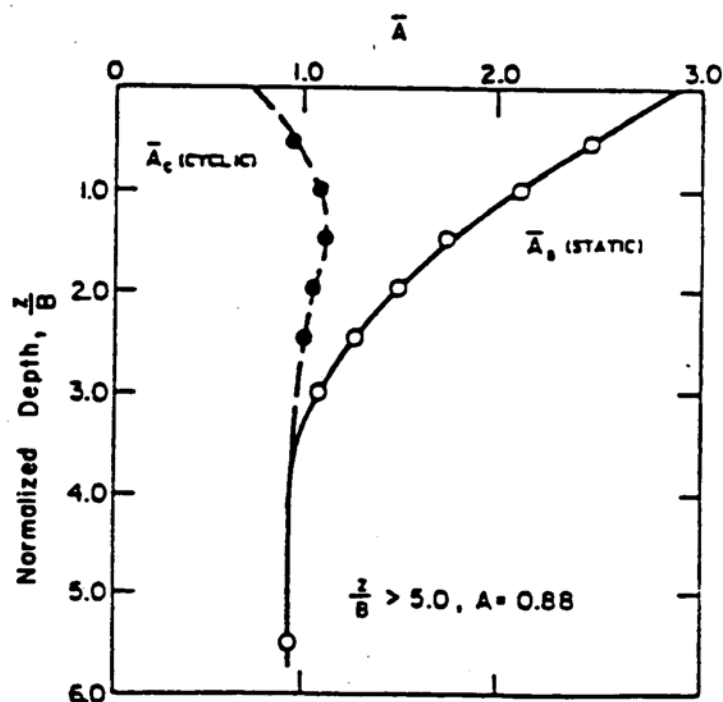
$$p_m = B_c p_s$$

Use the appropriate value of B_s or B_c from Figure 2.3.1.1.1.5-3 for the particular nondimensional depth, and for either the static or cyclic case. Use the appropriate equation for p_s . The two straight-line portions of the p-y curve beyond the point where y is equal to $B/60$ can now be established.

- (8) Establish the initial straight-line portion of the p-y curve.

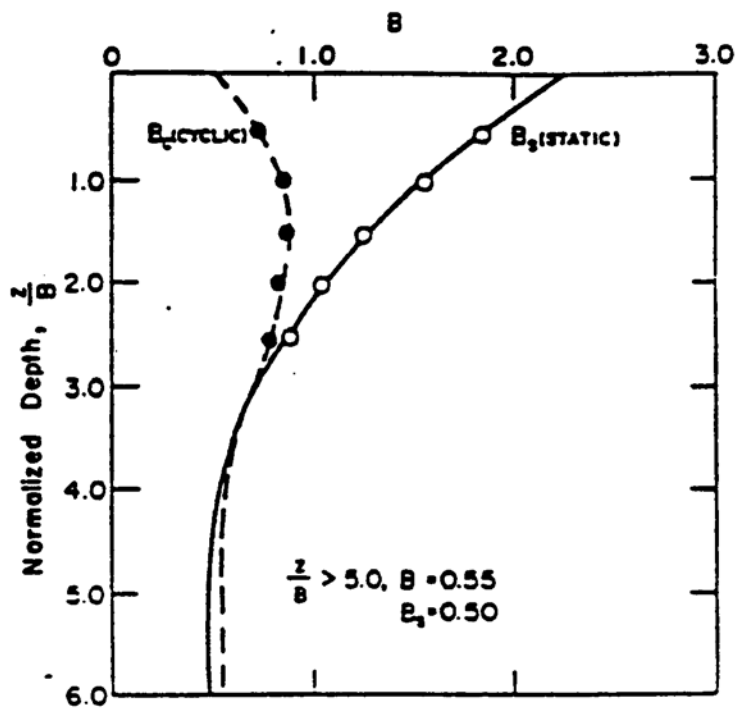
$$p = n_h z y$$

Use Table 2.3.1.1.1.5-2 to select an appropriate value of n_h .



Source: Adapted from Reference 5

Figure 2.3.1.1.1.5-2 - Nondimensional Coefficients $\bar{A}_c$ and $\bar{A}_s$



Source: Adapted from Reference 5

Figure 2.3.1.1.5-3 - Nondimensional Coefficient B

Table 2.3.1.1.1.5-1 - Nondimensional Coefficients for p-y Curves for Sand

Angle of Friction ϕ	$S_1^{(1)}$	$S_2^{(2)}$	$S_3^{(3)}$	z_t/B
(deg)	(dim)	(dim)	(dim)	(dim)
25	2.05805	1.21808	15.68459	11.18690
26	2.17061	1.33495	17.68745	11.62351
27	2.28742	1.46177	19.95332	12.08526
28	2.40879	1.59947	22.52060	12.57407
29	2.53509	1.74906	25.43390	13.09204
30	2.66667	1.91170	28.74513	13.64147
31	2.80394	2.08866	32.51489	14.22489
32	2.94733	2.28134	36.81400	14.84507
33	3.09732	2.49133	41.72552	15.50508
34	3.25442	2.72037	47.34702	16.20830
35	3.41918	2.97045	53.79347	16.95848
36	3.59222	3.24376	61.20067	17.75976
37	3.77421	3.54280	69.72952	18.61673
38	3.96586	3.87034	79.57113	19.53452
39	4.16799	4.22954	90.95327	20.51883
40	4.38147	4.62396	104.14818	21.56704

Notes:

$$^{(1)} S_1 = (K_p - K_a)$$

$$^{(2)} S_2 = \tan \beta (K_p \tan \alpha + K_o [\tan \phi \sin \beta (\sec [\alpha + 1]) - \tan \alpha])$$

$$^{(3)} S_3 = K_p^2 (K_p + K_p \tan \phi) - K_a$$

Source: Adapted from Reference 5

Table 2.3.1.1.1.5-2 - Representative Values of n_h for Sand

Relative Density D_r (%)	n_h (kcf)	
	Sand Below Water Table	Sand Above Water Table
< 35	35	45
35 - 65	100	155
> 65	215	390

Source: Adapted from Reference 5.

- (9) Establish the parabolic section of the p-y curve.

$$p = \bar{C}y^{1/n}$$

Fit the parabola between points k and m as follows:

- Determine the slope of line between points u and m as follows:

$$m = \frac{p_u - p_m}{y_u - y_m}$$

- Obtain the power of the parabolic section by

$$n = \frac{p_m}{m y_m}$$

- Obtain the coefficient $\bar{C}$ as follows:

$$\bar{C} = \frac{p_m}{y_m^{1/n}}$$

- Determine point k as

$$y_k = \left(\frac{\bar{C}}{N_h z} \right)^{n/(n-1)}$$

- Compute appropriate number of points on the parabola by using the first equation in (9) above.

Note: The step-by-step procedure is outlined and Figure 2.3.1.1.1.5-1 is drawn as if there is an intersection between the initial straight-line portion of the p-y curve and the parabolic portion of the curve at point k. However, in some instances there may be no intersection with the parabola. The equation in (8) above defines the p-y curve until there is an intersection with another branch of the p-y curve; or, if no intersection occurs, this equation defines the complete p-y curve. The soil-response curves for other depths can be found by repeating the above steps for each desired depth.

(b) Recommended Soil Tests

Triaxial compression tests are recommended for obtaining the angle of internal friction of the sand. Confining pressures shall be used which are close or equal to those at the depths being considered in the analysis. Tests shall be performed to determine the unit weight of the sand. In many instances, however, undisturbed samples of sand cannot be obtained, and the value of ϕ must be obtained from correlations with static cone penetration tests or from dynamic penetration tests.

Theoretical soil mechanics and engineering experience provide the basis for developing p-y curves. Actual methods of constructing such curves arise from duplication of load-deformation behavior observed during lateral load tests. The methods are typically generalized through correlations with soil properties, shaft diameter, and depth for broader application.

Because it is difficult to determine an exact percent of fixity, and because increased shaft head fixity results in decreased ground deflections, percent fixities somewhat less than those expected to be achieved are applied in deflection analysis of laterally-loaded drilled shafts so that the results obtained are somewhat conservative. It should be noted that the moment conditions at the two extremes of head fixity present the worst cases for consideration of the structural requirements of the shaft.

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PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX G - INTEGRAL ABUTMENTS

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1.1 PURPOSE

These guidelines establish design criteria for Integral Abutments. Construction details are presented on Standard Drawing BD-667M.

The Department has developed a spreadsheet, entitled *PennDOT's Integral Abutment Spreadsheet*, to aid engineers in the design of integral abutments. The spreadsheet includes the design criteria described herein and is based on the construction details shown on Standard Drawing BD-667M. The file can be downloaded from the Bridge Design and Technology Division's website at:

<ftp://ftp.dot.state.pa.us/public/Bureaus/design/bqad/Brid/SampleSpreadsheets/IntegralAbutment.xls>

1.2 DESIGN CRITERIA**1.2.1 Bridge Length**

Maximum allowable bridge total length, for 90° skew, measured between the centerlines of end bearings shall be taken as:

Concrete Structures = 590 ft.

Steel Structures = 390 ft.

Use of integral abutments on structures with lengths over the above limits shall be considered on case-by-case basis and shall require the written approval of the Chief Bridge Engineer at the Type, Size and Location stage.

Expansion bearings shall be eliminated wherever possible. Span arrangement and interior bearing selection shall be such that approximately equal movements take place at both abutments.

The ratio between the span lengths in the bridge shall be chosen such that no net negative reaction is produced at the

COMMENTARY

C1.1

Integral abutment bridges (Jointless Bridges) serve to accomplish the following desirable objectives:

- Long-term serviceability of the structure
- Minimal maintenance requirements
- Economical construction
- Improved aesthetics and safety considerations

A jointless bridge concept is defined as any design procedure that attempts to achieve the goals listed above by eliminating as many expansion joints as possible from the structure. The ideal jointless bridge, for example, contains no expansion joints in the superstructure, substructure or deck.

Integral abutment bridges are generally founded on one row of flexible piles made of steel or concrete. This permits the elimination of expansion joints, bearings, piles for horizontal earth loads and other uneconomical details.

When expansion joints are completely eliminated from a bridge, thermal stresses must be relieved or accounted for in some manner. The integral abutment bridge concept is based on the assumption that due to the flexibility of the piles, thermal stresses are transferred to the substructure by way of a rigid connection, i.e. the uniform temperature change causes the abutment to translate without rotation. The concrete abutment contains sufficient bulk to be considered a rigid mass. A positive connection to the girder ends is generally provided by encasement in the reinforced concrete end diaphragm. This provides for full transfer of forces due to thermal movements and live load rotational displacement to the abutment piles.

C1.2.1

Length limits for integral abutment bridges are a function of local soil and weather conditions. At the present, comprehensive design guidelines that take all design parameters into account are nonexistent in most states. Such guidelines would include design criteria concerning piles, approach slabs, wingwalls, backfill, drainage provisions, and the safe length limits of integral abutment bridges based on soil and weather profiles.

Based on past experience with jointless bridges in Pennsylvania, bridges within the specified range did not have any serious problems. Some other states have successfully constructed longer jointless bridges. When economically feasible, the use of jointless bridges with lengths exceeding the limits specified herein should be investigated.

The live load negative reaction at the abutments of bridges with short end spans may be relatively high. Transverse and

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abutment at any limit state.

1.2.2 Skew

Minimum allowable skew angle is:

- 70° for single spans in excess of 130 ft. and for multiple span structures
- 60° for single spans in excess of 90 ft. but not longer than 130 ft.
- 45° for single spans 90 ft. or less

1.2.3 Horizontal Alignment and Bridge Plan Geometry

Only straight beams may be used. Curved superstructures utilizing straight beams may be used, if approved by the Chief Bridge Engineer. All beams in each span of a curved bridge shall be parallel to each other. Integral abutments shall be allowed for straight bridges with splayed girders when the difference between the length of the two abutments does not exceed 10%.

1.2.4 Grade

The maximum vertical grade between abutments shall be 5%.

1.2.5 Scour

For structures over stream crossings, conduct an investigation for scour in accordance with guidelines given in Policies and Procedures, Chapter 7 (PP7). Utilize rip rap, geotextile and casing details per Standard Drawing BD-667M.

1.2.6 Drainage

The area behind the abutments shall be backfilled in accordance to Ap.G1.2.9 and a proper drainage system shall be provided (see Standard Drawing BC-751M for drainage details).

1.2.7 Loads and Load Combinations**1.2.7.1 Permanent Loads**

Permanent loads on the abutments include the weight of the girders, deck and approach slab, attached wingwalls,

COMMENTARY

uplift wind forces also produce negative forces acting on some of the piles. Due to the relatively light weight of integral abutments, it is possible to have a negative factored net force on some piles of bridges with short end span. To avoid any problems with the pile-abutment connection, a negative net force is not allowed on any pile.

C1.2.2

Earth pressure acts in a direction perpendicular to the abutments. For skewed bridges, the earth pressure forces on the two abutments produce a torque that causes the bridge to twist in plan. Limiting the skew angle reduces this effect. For continuous bridges, the mild skew angles reduce the forces acting on intermediate bents. For simple spans, limiting the length reduces the torque and the resulting twist of the bridge.

C1.2.3

Curved beams are not allowed to guard against the possibility of flange buckling caused by the beams trying to expand between the restraining abutments.

C1.2.5

Even when the piles are properly designed to withstand the loss of lateral support caused by scour, there is some concern for the integrity of bridge approaches. In light of this concern, the effect of scour on the stability of bridge approach embankment should be investigated.

C1.2.6

Providing a proper granular backfill and drainage system behind the abutment beneath the approach slab eliminates the build up of hydrostatic pressure and controls erosion of the abutment embankment.

C1.2.7.1

The dead loads on the abutments are distributed fairly uniformly across the width of the bridge. It is sufficiently

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intermediate diaphragms and the abutment end diaphragm. Other permanent loads on the first span and the approach slab such as wearing surface, barriers, utilities, sign structures, lighting systems and sound barriers shall also be considered. The dead loads on the first span of the bridge shall be determined using the dead load reaction of the girders calculated during the design of the superstructure. The dead loads on the abutments shall be distributed equally to all piles.

1.2.7.2 Live Loads

For the design of the abutment and the piles, live loads will be assumed equally distributed to all girders in the cross section. No multiple presence factors will be applied.

The total live load on the abutment shall be determined assuming the largest number of traffic lanes that may be allowed by the total bridge width as specified in A3.6.1.1.1. For bridges with sidewalks, the following two load cases shall be investigated:

1. The sidewalk is assumed to be eliminated. The number of traffic lanes is calculated based on the total width of the bridge including the width of the shoulders and sidewalks.

COMMENTARY

accurate to assume equal distribution of dead loads to all piles.

C1.2.7.2

Due to the high rigidity of the abutment beam, the loads are expected to be distributed to more piles than those directly under the load. The loads are likely to be distributed equally to all piles, particularly in the case of narrow bridges. For wide bridges, where the total length-to-depth ratio of the abutment beam is relatively high, loads applied near the edge of the bridge will not be distributed equally to all piles. However, the critical load case for the piles is expected to be the case of maximum load on the bridge, i.e. all traffic lanes and sidewalks are loaded. In this case, the loads are distributed across the width of the bridge and is expected to produce approximately equal loads on all piles. The elimination of the multiple presence factor will eliminate the possibility of underestimating the maximum pile load for wide bridges.

For the design of integral abutments, the live load reaction of any girder may be calculated as follows:

$$LLR_{IA} = \frac{LLR_{DP} * NL}{DF_{DP} * NG}$$

Where:

- LLR_{IA}: LL girder reaction for integral abutment design (kips)
- LLR_{DP}: LL reaction for interior girders from the girder design program (kips)
- DF_{DP}: LL reaction distribution factor from the girder design program
- NG: Number of girders in the cross sections
- NL: Maximum number of traffic lanes allowed by bridge clear width

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2. The number of traffic lanes is calculated based on the total width of the bridge excluding the sidewalks (i.e. the number of traffic lanes is calculated based on the clear width between curbs). For this load case, pedestrian loads acting on the abutment shall be calculated using Equation 1.2.7.2-1:

$$SWL = \frac{PL * W_{SK} * (L_{ES} - L_{AS})}{2} \quad (1.2.7.2-1)$$

SWL: Sidewalk reaction at the abutment (kips)

PL: Pedestrian load per unit area as specified in A3.6.1.6 (ksf)

W_{SK} : Total Width of sidewalk(s) (ft.)

L_{ES} : Length of end span of the bridge (ft.)

L_{AS} : Length of the approach slab (ft.)

Centrifugal force shall be applied to the abutments of curved bridge, subject to the limitations of Ap.G1.2.3. The magnitude and point of application of the centrifugal force shall conform to the requirements of A3.6.3.

Braking forces shall not be considered in the design of integral abutments.

Dynamic load allowance shall be considered in the design of both the abutment wall and the supporting piles.

In determining vehicular loads on the approach slabs, all possible traffic lanes shall be assumed to have the uniform lane load portion of the PHL-93 loading. The approach slab shall be assumed to act as a simple beam supported at one end of the abutment and supported at the other end on the sleeper slabs.

1.2.7.3 Wind Loads

Wind uplift on the superstructure and transverse wind loads on the superstructure and on live load shall be considered in the design. The magnitude and point of application of these forces shall be calculated according to A3.8 and D3.8.

The direction of transverse wind forces shall be taken perpendicular to the longitudinal axis of the bridge.

1.2.7.4 Thermal Movements

The change in the bridge length due to uniform temperature change, Δt , shall be calculated as:

$$\Delta L = \alpha \Delta t L \quad (1.2.7.4-1)$$

Where:

COMMENTARY

Equation 1.2.7.2-1 assumes that both the end span and approach slab are simply supported at both ends. The sidewalk load is generally small compared to traffic loads. Ignoring the continuity of the end span of continuous bridges is an acceptable approximation for the design of integral abutment

Braking forces are not considered in determining the lateral earth pressure on the abutment. Braking forces will conceivably push the abutment against the backfill causing the earth pressure to increase. However, the maximum earth pressure will never exceed the full passive pressure. The design of abutment components is based on the full passive pressure and, therefore, braking forces are not included.

Since the piles are attached directly to the structure, the dynamic load allowance is required to be considered in the pile design.

The truck load in each lane is assumed to be on the bridge and is considered in the girder reaction.

C1.2.7.3

The magnitude of wind forces due to wind pressure on the substructure is often small, relative to wind on the superstructure and on live loads, and may be neglected in the analysis.

For the case of curved bridges, subject to the limitations of Ap.G1.2.3, the direction of longitudinal and transverse wind forces on the end span of the bridge shall be taken parallel and perpendicular to the girders of the end spans, respectively.

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- L: Total bridge length between the centerlines of the abutments (in.)
- α : Coefficient of thermal expansion of the bridge material as specified in D5.4.2.2 and A6.4.1 for concrete and steel bridges, respectively (in/in/° F)
- Δt : Uniform temperature change (° F)
- ΔL : Change in bridge length due to uniform temperature change Δt (in.)

For the purpose of determining the maximum thermal movement at the abutments in one direction, $\Delta_{\max}$, after the time the connection between the abutment and the superstructure is made, the following uniform temperature changes, $\Delta t_{\max}$, shall substitute Δt in Equation 1.2.7.4-1.

- For steel bridges:
 $\Delta t_{\max} = 100^\circ \text{ F}$ temperature rise or fall
- For concrete bridges:
 $\Delta t_{\max} = 80^\circ \text{ F}$ temperature rise or fall

Where:

$\Delta t_{\max}$: The design uniform temperature rise or fall, equal to the maximum possible difference between the construction temperature and the extreme temperature of the bridge during its useful life (°F)

For the purpose of determining the total change in the bridge length between the extreme high and low temperatures during the life span of the bridge, the uniform temperature change, Δt_{total} , shall substitute Δt in Equation 1.2.7.4-1.

Where:

Δt_{total} : the sum of the temperature rise and temperature fall specified in D3.12.2.1 for the bridge material.

For simple spans with constant width, and for continuous spans having constant width and with both superstructure and substructure symmetric in the bridge elevation, the thermal movements at each abutment of bridges with integral abutments at both ends shall be taken as half the change in bridge length due to uniform temperature change.

COMMENTARY

The design uniform temperature rise or fall, $\Delta t_{\max}$, used in determining the maximum movement in one direction (after making the connection between the abutment and the superstructure) is meant to be an upper-bound for the following two cases:

- The difference between the extreme high temperature during the life span of the bridge and the lowest allowable construction temperature
- The difference between the extreme low temperature during the life span of the bridge and the highest allowable construction temperature

Using the specified design uniform temperature rise or fall, $\Delta t_{\max}$, will make the actual thermal movement always less than or equal to the design thermal movement regardless of the construction temperature.

The total range of thermal movement is used in the pile ductility check. This total range of thermal movement is not a function of the construction temperature. It is a function of the difference between the extreme low and high temperatures of the structure during its useful life.

In some cases, the bed rock may be too close to the surface at one end of the bridge that the use of piles may not be feasible. If approved by the Chief Bridge Engineer, a concrete footing on rock with a semi-integral abutment may be used at one end of the bridge and an integral abutment at the other end. The smaller depth of the semi-integral abutment relative to the integral abutment will produce smaller earth pressure on the semi-integral end of the bridge. This will cause most of the thermal movements to take place at the semi-integral end. For design purposes, the full change in bridge length will be assumed to take place at the semi-integral end of bridges with one integral abutment and one semi-integral abutment. Bridges with one integral abutment and one semi-integral abutment will usually be short bridges due to the limitations of maximum movements of semi-integral abutments.

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For other bridges, the thermal movements of the abutments due to uniform temperature change shall be determined taking into account the type of bearings at intermediate supports and the stiffness of the substructure including the piers, intermediate bents and the abutments.

The following approximate procedure should be used to determine the stiffness and estimated maximum thermal movement of the abutments:

1. Estimate the thermal movement of one abutment, $\Delta_{\max}$ (in.), based on the total length of the bridge and the design uniform temperature rise or fall, $\Delta t_{\max}$, for the bridge material. As a first step, this value may be assumed half the total change in the bridge length due to uniform temperature rise or fall, $\Delta t_{\max}$.
2. Determine the relative displacement ratio $\Delta_{\max}/H$, where H is the total height of the abutment, i.e. the sum of the heights of the pile cap, bearing pad, beams, haunch and deck slab (in.).
3. Determine the coefficient of passive earth pressure, k_p , corresponding to the calculated $\Delta_{\max}/H$ as:

$$k_p = 0.5 + 10.4 \frac{\Delta_{\max}}{H} \leq 3.0 \quad (1.2.7.4-2)$$

4. Determine the component of the total earth pressure force on the abutment parallel to the longitudinal axis of the bridge, F, using Equation 1.2.7.4-3.

$$F = 1/2 \gamma k_p H^2 L \sin \theta \quad (1.2.7.4-3)$$

Where:

γ : the soil density (kcf)
 θ : Skew angle (DEG)
H: Abutment height (ft.)
L: Length of the abutment along the skew (ft.)

5. Determine the stiffness of the abutment, $k = F/\Delta_{\max}$ (k/in)
6. Similarly, calculate the stiffness of the second abutment. The sum of the movements of the two abutments should equal the change in bridge length calculated for a uniform temperature change, $\Delta t_{\max}$, using Equation 1.2.7.4-1.

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Similarly, if the bridge has one or more expansion joints, the entire thermal change in length of the bridge unit starting at an integral abutment and ending at an expansion joint will be assumed to take place at the expansion joint.

The stiffness of the abutment is a function of the depth of the abutment, the thermal movement and the backfill type and compaction. Due to the nonlinear change of the coefficient of earth pressure, the stiffness of the abutment varies nonlinearly depending on the magnitude of the displacement.

Equation 1.2.7.4-2 is based on a two-line approximation of the passive pressure coefficient curve for loose sand in Figure C1.2.7.4-1. It assumes that the coefficient of earth pressure changes linearly from the at rest coefficient of 0.5 for no displacement to 3.0 when $\Delta t/H$ is equal to or greater than 0.02.

Figure C1.2.7.4-1 is from Barker (1997).

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7. Use a structural analysis computer program acceptable to the Department to determine a more accurate value of the abutment displacement, $\Delta_{\max}$, for the specified design uniform temperature rise or fall, $\Delta t_{\max}$. The abutment stiffness shall be modeled as a horizontal spring with a stiffness equal to that calculated for the abutment. The end of the bridge at the abutment shall be modeled as being supported on a vertical movable support. Alternatively, the abutment may be modeled as a single vertical column fixed at the bottom and pinned to the superstructure. The column shall have a stiffness equal to the stiffness of the abutment calculated in step 5 above. The length, L_c , the modulus of elasticity, E_c , and the moment of inertia, I_c , of the column shall be chosen to satisfy:

$$k = \frac{3E_c I_c}{L_c^3} \quad (1.2.7.4-4)$$

8. Compare the estimated displacement used in Step 1 to the displacement calculated in Step 7. If the difference is within 10%, there is no need for further iterations. If larger difference is obtained, go back to Step 2 and use the calculated displacement to determine a new value of $\Delta_{\max}/H$.

The total range of movement of an integral abutment during the bridge life span, Δ_{total} , (in.) shall be determined as:

$$\Delta_{\text{total}} = \Delta_{\max} (\Delta t_{\text{total}} / \Delta t_{\max}) \quad (1.2.7.4-5)$$

Where:

$\Delta_{\max}$: The maximum design thermal movement in one direction as calculated in Step 8 above (in.)

Due to the approximate nature of the process, a 10% difference between the estimated displacement (Step 1) and the calculated displacement (Step 8) is considered to be an acceptable difference.

The total range of thermal movement, Δ_{total} , is used in the pile ductility check. As shown in Steps 1 through 8, the movement of each abutment as a percentage of the total thermal change of length is a function of the stiffness of the abutment. Equation 1.2.7.4-1 assumes that the stiffness of the abutment used to calculate the maximum thermal movement in one direction will remain constant as the bridge moves from the extreme high temperature to the extreme low temperature. Ignoring the change in the abutment stiffness is an approximation used to simplify the design. The effect of this approximation is expected to be minor due to the small difference between Δt_{total} and $\Delta t_{\max}$.

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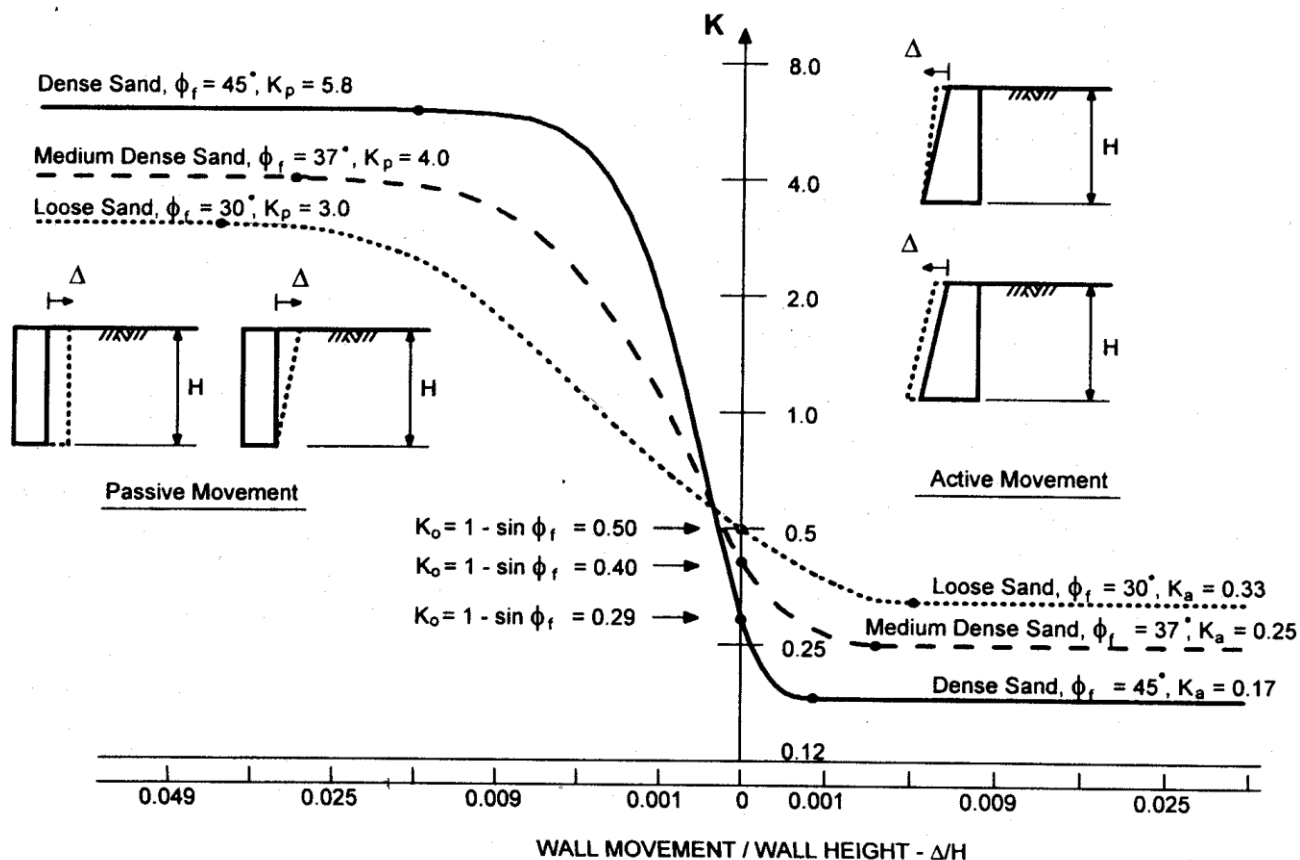


Figure C1.2.7.4-1 Passive Pressure Coefficient Curves

1.2.7.5 Secondary Loads

Except for the effect of creep and shrinkage on the vertical reactions of simple prestressed spans made continuous for live loads, abutment loads caused by creep, shrinkage, thermal gradient and differential settlements need only be considered for bridges longer than those specified in Ap.G1.2.1, which are approved by the Chief Bridge Engineer. The method of applying secondary loads requires the approval of the Chief Bridge Engineer.

1.2.7.6 Load Combinations

Load combinations for integral abutments shall be taken as shown in Table 1.2.7.6-1.

C1.2.7.5

The states contacted during the development of these specifications, which utilize integral abutment bridges, do not consider secondary loads in the design of integral abutment bridges with total length within the limits stated in Ap.G1.2.1. For longer bridges, the effect of secondary loads may be significant and are required to be considered in the design.

Table 1.2.7.6-1 – Load Combinations for Integral Abutments

Load Factors for Integral Abutments														
Load Case	SERV I		STR I		STR IP		STR II		STR III		STR V		Extreme II	
	max	min	max	min	max	min	max	min	max	min	max	min	max	min
γ_{DC}	1.00	1.00	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90	1.25	0.90
γ_{DW}	1.00	1.00	1.50	0*	1.50	0*	1.50	0*	1.50	0*	1.50	0*	1.50	0*
γ_{EV}	1.00	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00	1.35	1.00
γ_{EH}^{**}	1.00	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
γ_{ES}	1.00	1.00	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
γ_{LS}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50	0.50
γ_{LLIM}	1.00	0 [#]	1.75	0 [#]	1.35	0 [#]	1.35	0 [#]	0.00	0.00	1.35	0 [#]	0.50 ⁺	0.50 ⁺
γ_{PL}	0.00	0.00	0.00	0.00	1.75	1.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
γ_{WS}	0.30	0.30	0.00	0.00	0.00	0.00	0.00	0.00	1.40	1.40	0.40	0.40	0.00	0.00
γ_{WL}	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	1.00	0.00	0.00
γ_{CE}	1.00	1.00	1.75	1.75	1.35	1.35	1.35	1.35	0.00	0.00	1.35	1.35	0.50	0.50
γ_{TU}^{++}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Design Vehicle	PHL – 93		PHL-93		PHL-93		P-82		---		PHL-93		PHL-93	

* For existing bridges where the future wearing surface has been already applied, use 0.65 for the minimum γ_{DW} .

** Use the specified load factors for active pressure. Use a load factor of 1.25 for passive pressure.

For a negative reaction on an abutment (uplift), use the maximum load factor.

+ Extreme Event II Limit State is used to check the piles after scour takes place. A reduced live load is assumed on the bridge during the time between scour taking place and the time the bridge is closed to traffic.

++ Thermal movements is a major source of loads on the abutment and abutment piles. Both the passive earth pressure on the abutment and the stresses in steel piles due to thermal movements are not reduced by the plastic flow of the concrete expected due to the seasonal nature of the thermal movements. Therefore, no reduction in the load factor for uniform temperature is allowed and a load factor of 1.0 is used all the time. For concrete-filled pipe piles, the same load factor is applied. The effect of the plastic flow of the concrete filling is considered in the analysis by applying a reduction factor of 0.4 to the transformed area and moment of inertia of the concrete filling.

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1.2.8 Girder Types, Maximum Depth and Placement

Only steel I-beams, concrete I-beams and concrete spread box beams shall be allowed for use with integral abutments.

Maximum girder depth for use with integral abutments shall be 72 in.. Deeper girders may be used in integral abutment bridges if approved by the Chief Bridge Engineer.

All girders in integral abutment bridges, including box beams, shall be placed with their webs vertical. The roadway cross slopes and superelevation shall be accommodated by changing the depth of the concrete haunch across the width of the girders.

1.2.9 Backfill Material and Sequence

The area behind the abutments shall be backfilled with granular material such as O.G.S. (See RC-12M for backfill limits).

Flowable fill and large stone fill shall not be allowed in conjunction with integral abutments

A 1 in. thick sheet of preformed cellular polystyrene shall be placed against the entire area of the back face of the abutment below the bottom of the approach slab.

The fill located within a 2 ft. width directly behind both the abutment and the attached wingwalls shall be nominally compacted using two passes of a walk-behind vibratory-plate soil compactor. The fill in this area shall be compacted in 4 in. thick lifts. The fill behind both abutments shall be compacted simultaneously. The difference in the fill depth between the two ends shall not exceed 1 ft. at any point of time.

1.3 SUPERSTRUCTURE DESIGN

The superstructure shall be designed similar to conventional superstructures with expansion joints. The fixity developed as a result of rigidly connecting the superstructure to the abutments and the piers shall not be considered in the design of the superstructure.

Compressive axial load equal to the passive earth pressure on the abutment shall be considered in the design of the superstructure.

When flexibility for locating substructures is available, it is desirable to provide span ratios (interior/exterior) that produce near equal total design moments for all spans.

Deeper abutments are subjected to larger earth pressure force and, therefore, less flexible. Girder depth limit is based on past successful practices by other states and is meant to ensure a reasonable level of flexibility of the abutment. Deeper girders may be allowed when the soil conditions are favorable and the total length of the bridge is shorter than the length limits in Ap.G1.2.1.

Placing the girders with their webs vertical will allow the beam seat area to be horizontal in all cases and will facilitate using thin neoprene bearing pads.

C1.2.9

Flowable fill and large stone fill are self compacting. They will reduce the desired flexibility of the abutment.

The specified compaction is meant to reduce the passive earth pressure acting on the abutment and the wingwalls when moving toward the soil.

Filling behind both abutments simultaneously will keep the earth pressure on both abutments approximately equal at all times during construction. This will minimize unanticipated movements of the bridge due to imbalanced earth pressure forces.

C1.3

This provision will only be used to check the compressive stress at the ends of deep, widely spaced prestressed beams (beam depth greater than 6 ft. and spacing greater than 12 ft.. The estimated additional compressive stress is in the order of 30 ksf. For sections not close to the end of the span, the design under all loads is usually controlled by tension in the concrete and the additional compressive stress will not be a critical factor.

This preferred arrangement of span lengths results in more economical structures. The resulting length of end spans is approximately 80% of the adjacent interior spans. This length ratio produces relatively small negative live load reaction, thus,

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1.4 INTEGRAL ABUTMENT DESIGN**1.4.1 General**

The portion of the deck slab within 4 ft. from the front face of the abutment shall be poured after the remaining of the deck is poured. The end portion of the deck and the end diaphragm of the abutment will be poured at the same time.

Integral abutments shall be constructed in two stages:

Stage 1:

A pile cap supported on one row of vertical piles shall be constructed. The top of the pile cap shall reach the bottom of the bearing pads under the girders. The top of the pile cap shall be smooth in the area directly under the girders and a strip 2 in. wide around this area. Other areas shall be intentionally roughened (rake finished).

Stage 2:

After pouring the entire deck slab, except for the portions of the deck within 4 ft. from the front face of the abutments, an end diaphragm encasing the ends of the bridge girders shall be poured. The end 4 ft. of the deck shall be poured simultaneously with the end diaphragm for beams ≤ 36 in.. For beams deeper than 36 in., wait a minimum of 2 hours after casting the end diaphragm before casting the deck slab portion over the end diaphragm. This ensures that settlement of the poured concrete has occurred in the end diaphragm placement before beginning placement of the deck slab portion over the end diaphragm. The concrete end diaphragm and the end portion of the deck shall be poured when the surface temperature is between 35° F and 89° F. The expected air temperature during the six hours following pouring this concrete should be within the same range. The end diaphragm shall have the same width as the pile cap constructed in Stage 1. The end diaphragm shall extend from the top of the pile cap to the top surface of the deck slab.

The width of the pile cap shall be 4 ft..

The height of the pile cap shall be 3'-3" at its shallowest point.

The maximum difference between the minimum and maximum depth of the pile cap shall not exceed 1 ft. for skews less than 80° and 1'-6" for skews 80° or larger.

reduces the possibility of negative net reaction at the abutments.

C1.4.1

Pouring most of the deck slab before pouring the connection between the integral abutment and the superstructure minimizes the dead load rotations imposed on the piles. The dead load rotations at the ends of the girders will not be transferred to the piles. The end portions of the deck and the abutment wall (end diaphragm) may be poured immediately after the deck.

When a median barrier exists on the bridge, split the abutment at the center of the barrier and leave a 1 in. gap between the two halves. Place a closed cell neoprene sponge in the gap. Glue the neoprene sponge to both sides. Place an approved membrane waterproofing along the rear face of the abutment along the vertical joint.

The 4 ft. width can accommodate four bars of longitudinal reinforcement of the pile cap, concrete cover, stirrups, HP14 piles at 45° skew and allows for 3 in. of pile placement tolerance in any direction.

The height of the pile cap will vary along the length of the abutment due to differences in the beam seat elevations. These differences are due to the cross slopes of the roadway on the bridge and due to superelevation.

The bottom of the pile cap will generally be horizontal. If the limit on the maximum difference in pile cap depth could not be satisfied, the bottom of the abutment may be made

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1.4.2 Piles**1.4.2.1 General**

Integral abutments may be supported on end bearing piles or friction piles. Steel-encased concrete piles or steel H-piles may be used.

Monotube piles shall not be allowed in integral abutments.

Tapered pipe piles may be used provided the taper point is below the point of contraflexure.

The minimum edge distance between the centerline of the pile and the end of the abutment, measured along the skew, shall be the larger of:

- 1'-6" and
- the distance required to provide 3" clearance from the pile to the horizontal reinforcement extending from the wingwall.

The maximum edge distance between the centerline of the pile and the end of the abutment, measured along the skew, shall be the larger of:

- 2'-6" and
- the following formula (rounded up to the next 3" increment)

$$d_{\max} = \frac{W + d_{\text{pile}} / 2}{\sin \theta}$$

Where:

W: Width of wingwall at rear face of integral abutment, neglecting the 1' haunch (ft.)

d_{pile} : Outside diameter for pipe piles or pile depth for H-piles (ft.)

θ : Skew angle (DEG)

The piles shall be embedded 1'-6" into the abutment. The maximum pile spacing for integral abutment piles shall be 10 ft. The minimum pile spacing requirements of D10.7.1.2 shall apply.

parallel to the roadway surface resulting in a constant total height of the abutment.

C1.4.2.1

Steel-encased concrete piles (pipe piles) are less ductile than H-piles and may be damaged if subjected to relatively large lateral displacements and rotations. Thus, the analysis may prove they are inadequate for use in longer bridges, in particular when the girders in the end span are flexible. The ends of such girders will be subjected to a relatively high rotation due to live loads and composite dead loads. .

There is no available data on the behavior of monotube piles under plastic deformations. The thin walls, the taper and the wall corrugations of the monotube piles are expected to make their inelastic behavior different from that of smooth pipes. These factors make it likely that these piles will not have the ductility required to resist the movements of integral abutments without local buckling of the pile sections.

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Steel H-piles shall be driven with their weak axis perpendicular to the centerline of the beams of the end span regardless of the skew.

For structures less than or equal to 100 ft. in length, pre-augering is not required except for situations where shallow rock, rock embankments, or obstructions are encountered within 10 ft. of the bottom of pile cap, or unless otherwise required due to the geology of the location. For structures over 100 ft. in length, oversize pre-augered holes shall be drilled at pile locations prior to driving the piles. The minimum depth of the pre-augered holes shall be 10 ft.. The piles shall then be placed in the pre-augered holes. The holes shall be filled with dry sand or pea gravel after the piles are placed in the holes but before driving the piles. Following, the piles shall be driven to the required tip elevation, bearing stratum and driving resistance.

The minimum diameter of the pre-augered holes shall be the larger of:

- 2 ft.
- The largest dimension of the pile, i.e., pipe pile diameter or H-pile diagonal, plus 10 in..

Integral abutment piles shall be anchored to the abutment as follows:

- For pipe piles: A reinforcement cage shall be placed inside the top portion of the pile. The longitudinal bars shall be 4-#8 for 1'-0" diameter piles and 5-#8 for 1'-2" and 1'-4" diameter piles.
- For H-piles: One reinforcement bar, 3 ft. long shall be placed through a hole in the web of the pile. The hole diameter shall be 1.5 times the reinforcement bar diameter and shall be located 6 in. from the top of the pile.

The details of the pile anchorage are shown on Standard Drawing BD-667M.

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Thermal movements of integral abutments are parallel to the longitudinal axis of the girders of square bridges. Orienting the H-piles with their weaker axis perpendicular to the longitudinal axis of the bridge reduces the restraint forces developed at the pile-to-abutment connection due to thermal movements. In case of skewed and curved bridges, thermal movements are affected by the geometry of the bridge. However, especially for mild skews, the thermal movements are still close to be parallel to the girders of the end span. Therefore, H-piles are still required to be driven with the weaker axis perpendicular to the longitudinal axis of the girders of the end span.

Installing the piles in pre-augered oversize holes is a common practice used in the design of integral abutments. Horizontal soil forces act on the piles of integral abutments when subjected to thermal movements. The magnitude of the horizontal soil forces is a function of the type of soil and the magnitude of thermal movements. The use of pre-augered holes reduces the magnitude of the horizontal soil forces and moves the point of fixity lower in the soil. The deeper the point of fixity, the larger the radius of curvature of the piles when deformed by the effect of thermal movement. The larger radius of curvature reduces the ductility demand on the piles. Increasing the depth of the pre-augered holes should be considered when higher ductility of the piles is required; particularly when the top layers of the soil are stiff.

Pre-augered oversize holes may also be used to minimize downdrag forces when piles are driven in compressive soils. Excessive downdrag forces, if not prevented, may lead to a premature failure of the piles. The use of oversized pre-augered holes also tends to minimize the effects of concrete shortening due to creep effects when used with prestressed concrete beam superstructures.

Two conflicting concerns are to be considered when refilling pre-augered holes. First, pile buckling is to be prevented by providing lateral support along the full length of the pile and, second, the fill should be flexible enough to allow the pile to move laterally without being subjected to high lateral loads and moments. Sand or pea gravel fills provide the required characteristics.

Integral abutment piles are anchored in the abutment to guard against any unanticipated uplift on the piles. In addition, for the stiffer pipe piles, they ensure adequate moment transfer between the pile and the abutment.

The reinforcement bar size is based on the pile size and is shown on Standard Drawing BD-667M.

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All piles shall have a penetration length into the original soil at least equal to the larger of the following two values:

- (1) For piles through embankment fills, the minimum penetration into original ground specified in D10.7.1.3
- (2) The depth to point of fixity specified in D6.15.3.3 plus 5 ft.

The minimum acceptable pile length, measured from the bottom of the abutment to the pile tip, shall be 15 ft..

When scour is anticipated, the minimum pile length shall be provided beyond the depth of computed maximum scour.

1.4.2.2 Pile Design

Piles shall be designed for normal vertical loads and lateral loads. Piles shall also be analyzed for both the abutment thermal movements and the superstructure live load and composite dead load rotations. The thermal movements shall be calculated in accordance with Ap.G1.2.7.4.

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This minimum length is required to provide fixity and adequate capacity to the piles. Piles satisfying the required minimum length are expected to adequately resist vertical and lateral loads and avoid a stilt-type effect.

Article D6.15.3.3 requires COM624P or LPILE 5.0 analysis for the determination of the point to fixity.

In cases where the bedrock is close to the surface, the pre-augered holes shall be continued into the bedrock to provide the minimum required pile length.

The minimum pile length is provided below the design scour depth to ensure the safety of the structure.

C1.4.2.2

Larger pile reactions may be obtained by maximizing the transverse moments on the abutments. These transverse moments are produced by (1) transverse wind load on the structure and on live load, and (2) centrifugal force on curved bridges, subject to the limitations of Ap.G1.2.3.

Overturning moments due to wind load on the superstructure and on live load, wind uplift and centrifugal force transmitted to the abutment are assumed to be resisted by axial forces on the piles. The pile loads are assumed to increase linearly from zero at the center of the abutment to maximum for the piles near the end of the abutment as shown in Figure C1.4.2.2-1. For clarity, only centrifugal force and wind on the superstructure are shown in this figure. Wind uplift and wind on live load are applied similarly.

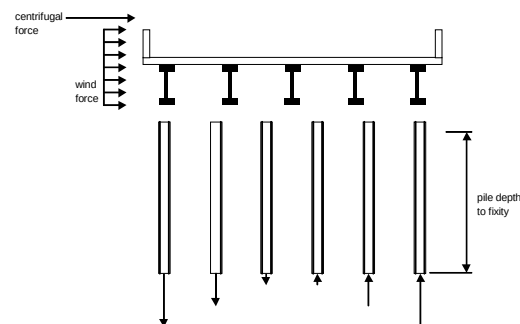


Figure C1.4.2.2-1. Pile Vertical Loads Due to Overturning Moments

Earth pressure on the abutments of continuous spans will produce additional vertical loads on the piles. The value of the additional vertical load per pile may be approximately calculated using Figure C1.4.2.2-2 and Equation C1.4.2.2-1. This approximate calculation assumes that the end span is separated from the remaining of the bridge and that the end of

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the span at the first pier is simply supported.

$$P_T = \frac{\left(\frac{2FH}{3N}\right) + H_T H + M_T}{L} \quad (C1.4.2.2-1)$$

Where:

P_T : The additional vertical load per pile due to earth pressure on the abutment (kips)

N : Number of piles in the abutment

F : Passive pressure force on the abutment (kips)

H : The height of the abutment (ft.)

H_T : The horizontal force on the pile, may be calculated as twice the plastic moment of the pile divided by the distance from the bottom of the abutment to the point of fixity of the piles (kips)

M_T : The moment at the top of the pile, may be conservatively taken as the plastic moment of the pile (kip-ft.)

L : Length of end span (ft.)

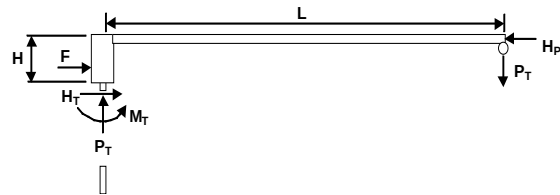


Figure C1.4.2.2-2 Pile Vertical Load Due to Earth Pressure on the Abutment

The design of the pile is controlled by the minimum capacity as determined for the following cases:

Case A: Capacity of the pile as a structural member according to the procedures outlined in A10.7.3.13, D10.7.3.13 and Ap.G1.4.2.4. The design for combined moment and axial force shall be based on an analysis that takes the effect of the soil into account.

Case B: Capacity of the pile to transfer load to the ground

Case C: Capacity of the ground to support the load as specified in A10.6.2.6, A10.7.3 and D10.7.3.

The top portion of friction piles where the lateral deflection exceeds 0.02 of the pile width perpendicular to the longitudinal axis of the bridge shall be ignored in determining the friction capacity of the piles.

Maximum load capacity of the pile based on the soil conditions will be determined using the soil properties provided by the geotechnical engineers.

The movement of the pile reduces the effectiveness of its contact with the soil at its top portions. This effect diminishes when the pile movement is less than 0.02 of the pile width (Fleming et. al. 1985). The pile width should be taken as the H-pile depth or the diameter of round piles.

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1.4.2.3 Pile Analysis

When the piles in an abutment vary in length such that some or all of the piles have one point of inflection (zero moment), the analysis shall be conducted for both of the end piles in the abutment. For other cases, the analysis shall be conducted on the longer of the two end piles.

Moments, shears, and deflections along the length of the piles shall be determined using the computer program COM624P or LPILE 5.0 by specifying a top of pile boundary condition which permits a specified lateral deflection and an applied moment:

Apply the maximum vertical pile load to the top of the pile simultaneously with the abutment maximum thermal movement, Δ_{max} , calculated using Ap.G1.2.7.4. The axial load and deflection should be entered as positive values. Apply the negative plastic moment at the head of the pile and run the pile analysis using COM624P or LPILE 5.0 until one of the two conditions below is met:

1. If the calculated pile head rotation (positive value) is less than the end rotation of the beam due to live load and composite dead load, the analysis is complete.
2. If the calculated pile head rotation (positive value) is greater than the end rotation of the beam due to live load and composite dead load, iteratively reduce the applied moment at the head of the pile until the rotations are equal or within tolerance.

The soil properties within the depth of the pre-augered holes shall be assumed to be the weaker of loose sand and the surrounding soil.

In determining the plastic moment of concrete-filled pipe piles, the concrete inside the pipe shall be ignored.

For concrete-filled pipe piles, M_y used in the analysis shall not exceed M_p .

The moment of inertia of the concrete-filled pipes shall be taken as that of the transformed section (transformed to steel). The modulus of elasticity shall be taken as that of steel. A reduction factor of 0.4 shall be applied to the concrete area and moment of inertia when calculating the transformed section properties.

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C1.4.2.3

Extending the length of the piles below the second point of zero moment has a very little effect on the results in the upper, more critical, portions of the pile. In some cases, the bedrock elevation, and the length of piles, vary significantly and some or all of the piles will have only one inflection point.

The behavior in these cases may vary and it is required to perform the analysis for both of the end piles. The choice of the end piles is based on them having the highest load and that they are likely to be the longest and shortest piles in the abutment (assuming bedrock slope is approximately linear across the width of the bridge.)

The computer program COM624P or LPILE 5.0 is based on assuming elastic behavior. For small movements, the pile will remain elastic. Step 2 assumes elastic behavior as long as the maximum moment is below the plastic moment. Once the pile reaches its plastic moment, the moment transferred to the pile cannot be increased further.

Once the moment at the head of the pile reaches the plastic moment and a plastic hinge forms, the rotations of the pile head become independent of the rotations of the abutment. The difference between the rotation of the pile head obtained from COM624P or LPILE 5.0 analysis and the rotation of the superstructure represents the inelastic deformation of the plastic hinge at the top of the pile.

There is no simple way to use COM624P or LPILE 5.0 to analyze piles with maximum moment between the yield moment and the plastic moment. As a simplification, piles with maximum moments exceeding the yield moment but below the plastic moment are assumed to remain elastic. This will be a conservative solution since that COM624P or LPILE 5.0 results overestimate the moment in the piles when the moment exceeds the yield moment. This is because the program does not consider the redistribution of forces associated with inelastic behavior.

Representative k values for loose sand within the pre-augered hole are 20 lb/in.³ saturated (below the water table) and 25 lb/in.³ dry (above the water table).

The plastic rotations required at plastic hinge locations will cause the concrete filling to crack and become ineffective in resisting the applied plastic moments. At lower moments, the concrete will still be effective and should be considered in determining the area and moment of inertia of the section.

When the applied moment is less than the yield moment, the concrete filling is assumed effective. In some cases, M_y (calculated using Equation C1.4.2.3-4 below) may exceed M_p (calculated using Equation C1.4.2.3-1). This is not allowed as the plastic moment is assumed to be the upper bound of the section resistance.

Thermal movements of integral abutments are seasonal in nature. The relatively long time over which these movements take place allows for plastic flow of the concrete. The effective stiffness of the concrete in these cases is less than its elastic stiffness. The reduction factor of 0.4 applied to the concrete area and moment of inertia when calculated the

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transformed section properties is meant to account for this nonlinear behavior. The value of 0.4 is chosen to match that required by A6.9.5.1 when analyzing concrete-filled pipe columns.

The plastic moment, transformed area, transformed moment of inertia and yield moment of concrete-filled pipe can be calculated as follows:

$$M_p = 4f_y R^2 t \quad (C1.4.2.3-1)$$

$$A_t = 2\pi R t + \frac{0.4\pi R_i^2}{n} \quad (C1.4.2.3-2)$$

$$I_t = \pi R^3 t + \frac{0.1\pi R_i^4}{n} \quad (C1.4.2.3-3)$$

$$M_y = \frac{f_y I_t}{R_o} \quad (C1.4.2.3-4)$$

Where:

- R_i : The inside radius of the steel pipe (in.)
- R_o : The outside radius of the steel pipe (in.)
- R : the average radius of the steel pipe (in.)
- t : the thickness of the pipe (in.)
- f_y : the specified minimum yield stress for the pipe material (ksi)
- n : The modular ratio between the steel and the concrete filling
- M_p : The plastic moment of the steel pipe (kip-in.)
- M_y : The yield moment of the composite section (kip-in.)
- A_t : The transformed area of the steel and concrete section (in.²)
- I_t : The transformed moment of inertia of the steel and concrete (in.⁴)

Under normal conditions, a COM624P or LPILE 5.0 analysis shall be conducted assuming that the entire pile is below ground surface. When scour is expected, a separate analysis shall be conducted assuming that the length of the pile from the bottom of the abutment to the ground surface after scour takes place is unsupported.

1.4.2.4 Pile Design as a Structural Member

The portion of the pile between the bottom of the abutment and the closest point of zero moment shall be checked for ductility as specified in Ap.G1.4.2.5.

The piles shall be checked as a structural member subject to axial load and flexure. The axial force shall be taken as the full pile load and the moment shall be taken as the maximum moment within the length under consideration. The moments shall be obtained from the COM624P or LPILE 5.0 computer program output. The length used in determining the axial load

C1.4.2.4

The stresses in the pile near its connection to the abutment need not be checked. Yielding and redistribution of forces is allowed in this area.

For friction piles, the axial force decreases along the length of the pile. However, the first point of zero deflection is usually a short distance below the bottom of the abutment and the axial force at this point will essentially be equal to the pile load.

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capacity of the pile as a compression member shall be taken as:

1. The portion of the pile between the two points of zero moment closest to the bottom of the abutment. This length shall be assumed pinned at both ends.
2. For shorter piles that have only one point of zero moment, the length between the point of zero moment and the tip of the pile. This length shall be assumed pinned at both ends.
3. When scour is expected to expose the top portion of the piles, the length of the pile from the bottom of the abutment to the pile point of fixity should be considered unsupported. This length shall be assumed fixed at both ends. This case shall only be checked if the length from the bottom of the abutment to the first point of fixity is larger than 1.54 times the length used in checking the pile before scour (case 1 or 2 above, whichever was applicable). This case is applied at the extreme event limit state.

The scour case shall control when the exposed portion of the piles is long. The 1.54 factor is the ratio between the effective length factor for a member pinned at both ends, i.e. 1.0, and that of a member fixed at both ends, i.e., 0.65. Beyond the specified length ratio, the fixed-fixed compression member shall have a smaller capacity than the pinned-pinned member.

1.4.2.5 Pile Ductility Requirements

Integral abutment piles shall satisfy the following equations:

For steel H-piles:

$$2 \left[\frac{\Delta_{\text{total}}}{2L} - \frac{M_p L}{6EI} \right] + \theta_w \leq \frac{3C_i M_p L}{4EI} \quad (1.4.2.5-1)$$

$$C_i = \frac{19}{6} - 5.68 \sqrt{\frac{f_y}{E}} \frac{b_f}{2t_f} \quad (1.4.2.5-2)$$

For concrete-filled pipe piles:

$$2 \left[\frac{\Delta_{\text{total}}}{2L} - \frac{M_p L}{6EI} \right] + \theta_w \leq \frac{C_i M_p L}{2.08EI} \quad (1.4.2.5-3)$$

$$C_i = 3.5 - 1.25 \sqrt{\frac{f_y}{E}} \frac{D}{t} \quad (1.4.2.5-4)$$

Where:

- Δ_{total} : Total thermal movement of the abutments calculated by Equation 1.2.7.4-5 (in.)
- L: Twice the length from the bottom of the abutment to the first point of zero moment in the pile (in.)
- M_p : Plastic moment of the H-pile in weak axis bending or the plastic moment of the steel pipe without considering the concrete filling (kip-in.)
- E: Modulus of elasticity of the steel (ksi)
- I: H-pile moment of inertia about the weak axis or moment of inertia of the filled pipe considering both

C1.4.2.5

The basis for the ductility requirements for H-piles may be found in the Transportation Research Record Report No. 1223 and Greimann, et. al (1987). The ductility requirements for pipe piles were driven using the same principals.

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- the steel pipe and the concrete filling calculated using Equation C1.4.2.3-3 (in.⁴)
- θ_w : Maximum range (Positive + |negative|) of factored angle of rotation of the superstructure at the abutment calculated assuming the structure is simply supported on the abutment, always taken as a positive quantity. This rotation is the sum of the rotations due to composite dead loads and live loads. Live load rotations shall be calculated assuming all traffic lanes on the bridge are loaded and the live loads are distributed equally to all girders (rad).
- C_i : A ductility reduction factor for piles, $0.0 < C_i < 1.0$
- f_y : The specified minimum yield stress for pipe material (ksi)
- b_f : width of pile flange (in.)
- t_f : pile flange thickness (in.)
- D : Outer diameter of the concrete-filled pipe pile (in.)
- t : Thickness of concrete-filled pipe pile (in.)

1.4.3 Abutment/End Diaphragm/Pile Cap Design

The width of the abutment end diaphragm and pile cap shall be 4 ft..

The effect of cushion material, placed behind the end diaphragm of the abutment, on the earth pressure shall not be considered in the design.

The longitudinal bars in the deck slab shall be extended as far as practical into the end diaphragm. When the transverse bars in the deck are placed parallel to the abutment, these bars need not be placed past the front face of the end diaphragm. In case of skewed bridges with deck transverse bars placed perpendicular to the girders, the transverse bars of the slab shall be extended inside the end diaphragm and be terminated as close to the back face of the end diaphragm as practical.

The longitudinal reinforcement of the abutment shall be designed assuming that all vertical loads on the abutment are resisted by the bottom 3'-4" of the abutment; i.e. resisted by the pile cap. Abutment walls shall be designed for the following two cases:

- Case 1: Vertical loads assuming the abutment wall to act as a continuous beam supported on the piles. The effect of the lateral loads transmitted to the abutment shall be considered in determining the maximum girder vertical reactions.

COMMENTARY

C1.4.3

The earth pressure forces acting on the abutment due to thermal expansion of the bridge will be reduced by the preformed cellular polystyrene sheet placed behind the abutment before backfilling, as specified in Ap.G1.2.9. The preformed cellular polystyrene will yield under pressure and thus help in reducing the restraining forces.

The reinforcement details shown on Standard Drawing BD-667M are intended to satisfy these requirements.

In case of skewed bridges with deck transverse bars perpendicular to the girders, place the top reinforcement bars of the end diaphragm below the top reinforcement layer of the deck. This arrangement eliminates the interference between the end diaphragm reinforcement and the transverse deck slab bars extending inside the end diaphragm.

Loads acting on the abutments are listed in Ap.G1.2.7

Simplified approaches may be used to calculate shears and moments in the abutment walls. Continuous beam moments may be taken as 80% of simple span moments. Shears may be taken equal to simple span shears. Due to the relatively large depth and short spans of the abutment walls, minimum reinforcement is usually sufficient to satisfy the strength requirement.

In determining the maximum girder reaction, overturning moments due to wind load on the superstructure and on live load, wind uplift and centrifugal force are assumed to be transmitted to the abutment through vertical girder reactions. The girder vertical reactions due to lateral loads are assumed to increase linearly from zero at the center of the abutment to maximum for the exterior girders as shown in Figure C1.4.3-1.

SPECIFICATIONS

COMMENTARY

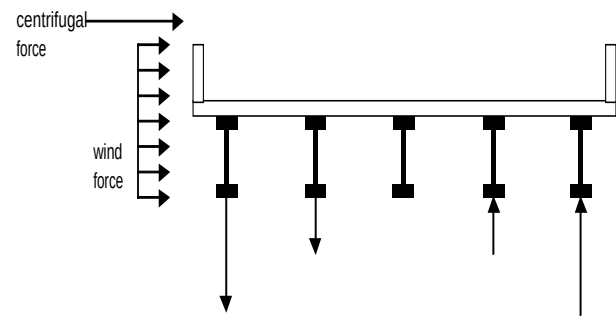


Figure C1.4.3-1 - Girder Vertical Reactions Due to Overturning Moments

Dead loads applied before the top part of the abutment hardens are resisted by the pile cap. Loads applied after that are resisted by the full abutment wall. For simplicity, all loads are assumed to be applied to the pile cap. Due to the large size of the pile cap, the increase in the reinforcement due to this conservative assumption is minor. Figure C1.4.3-2 gives the longitudinal reinforcement of the pile cap. The shown reinforcement represents an upper-bound for the required reinforcement assuming the girders are located at the positions that produce maximum effects on the pile cap and assuming a conservative value of other dead loads on the abutment wall.

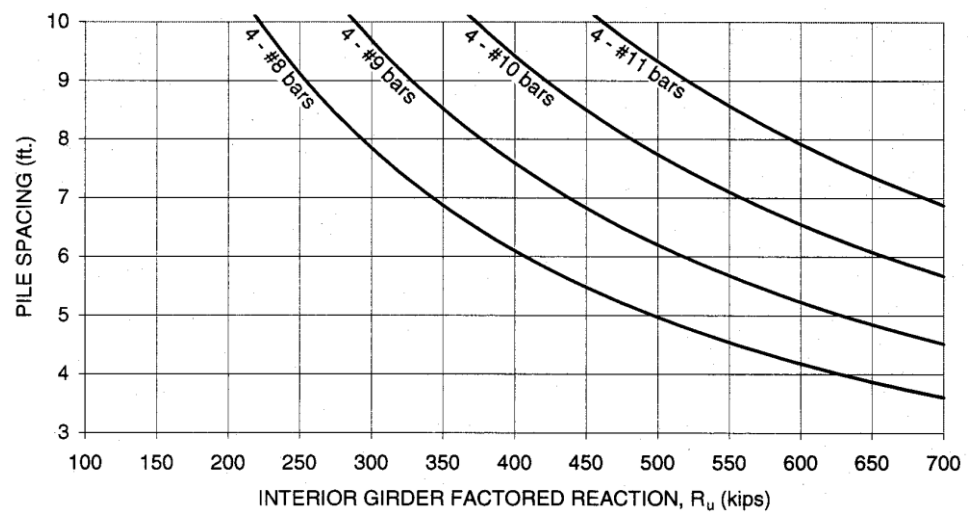


Figure C1.4.3-2 - Pile Cap Top and Bottom Reinforcement

Case 2: Horizontal earth pressure loads assuming the abutment to act as a continuous beam supported on the girders.

Stirrups designed to resist vertical shear forces acting on the abutment shall be provided.

The maximum possible earth pressure force on the abutment is the passive earth pressure. For abutments with relatively small thermal movement, the full passive pressure may not develop. The reinforcement shown on Standard Drawing BD-667M will be sufficient to resist the full passive pressure for all cases of integral abutments. Due to the large thickness of the abutment, this reinforcement is controlled by the minimum reinforcement requirements for most cases.

SPECIFICATIONS

L-shaped reinforcement bars shall be provided to transfer the connection moment between the abutment and the superstructure. The vertical leg of these connection bars shall be placed as close as practical to the back face of the abutment. The horizontal leg of these bars shall be extended into the deck at the elevation of the deck top longitudinal reinforcement for a minimum of 3 ft. beyond the front face of the integral abutment. The connection bars shall be #6 @ 9 in. for girders up to 8 ft. deep. For deeper girders, if such girders were approved by the Chief Bridge Engineer, the L-shaped bars shall be designed to transfer the maximum expected connection moment between the abutment and the superstructure.

1.4.4 Wingwalls

Only U wingwalls (wingwalls parallel to the longitudinal axis of the bridge) shall be used in conjunction with integral abutments.

A 1 ft. chamfer shall be used between the abutment and all wingwalls. The 1 ft. shall be measured from the point of intersection of the back face of the abutment and the wingwall.

Depending on the situation, one of the following three wingwall configurations shall be used:

1. Attached rectangular wingwall: This is the preferred wingwall configuration and shall be used where possible. The maximum length of the rectangular wingwall measured from the back face of the abutment shall be 8'-8".
2. Tapered attached wingwalls: Tapered wingwalls shall be used when the length of the wingwall exceeds the maximum length allowed for rectangular wings. The maximum length of the tapered wingwalls measured from the back face of the abutment shall be 16'-7". The depth of the wingwall at its free end shall be 2 ft.
3. Detached wingwalls: Detached wingwalls shall be used when the required length of the wingwall exceeds that allowed for attached wingwalls. A 3 ft. long attached rectangular wing wall shall be used and a compression seal expansion joint will separate the attached portion of the wingwall from the detached portion. The detached portion of the wingwall shall be designed as an independent retaining wall.

Wingwall reinforcement shall be taken as shown on Standard Drawing BD-667M.

COMMENTARY

In some cases in the past, transverse cracks in the deck were observed parallel to the integral abutments. This was attributed to the premature termination of the bars connecting the superstructure to the abutment. The provided reinforcement (#6 @ 9 in.) and their specified embedment in the deck are intended to satisfy the connection requirements for girders up to 8 ft. deep spaced at up to 13'-9" girder spacing.

C1.4.4

Wingwalls parallel to the abutment are subject to significant passive pressure and are not allowed to be integral with the abutment.

Rectangular wingwalls simplify construction but they are subject to high earth pressure. As the length of the wingwall increases, tapering the wings will reduce the earth pressure force and will allow the use of longer wings.

Skewed integral abutment bridges tend to twist in the horizontal plane. This results in some wingwalls moving toward the backfill. These movements are not expected to be large enough to produce passive pressure. However, due to the uncertainty of the magnitude of the movements, the wingwalls are checked for the full passive pressure.

SPECIFICATIONS

1.5 APPROACH SLAB

Approach slabs shall be provided at each abutment. Approach slabs shall be cast on two (2) layers of 4 mil. thick polyethylene sheets. The width of the approach slab perpendicular to the direction of traffic shall run from bridge gutter line to bridge gutter line. The approach slab skew angle shall match the bridge skew angle and be within the skew limitations listed in Ap.G1.2.2.

The approach slab shall be designed as a 25'-0" long simple span slab bridge subjected to all applicable loads. The soil support under the approach slab shall be ignored in the design. The thickness of the approach slab shall be 1'-6".

A contraction joint shall be located along the edge of the approach slab at the abutment. Form the joint and seal with an approved sealer. The vertical interface between the abutment and the approach slab shall be coated with an approved bond breaker prior to pouring the approach slab.

Use bidwell to finish the approach slab.

The approach slab shall be connected to the abutment using epoxy coated dowels extending from the end diaphragm through the bottom of the approach slab near the back face of the end diaphragm. These bars shall be #6 @ 9 in. and will be detailed as shown on Standard Drawings BD-667M and BD-628M.

The approach slab shall rest on the abutment at one end and on a sleeper slab at the other end. The reinforcement and dimensions of the approach and sleeper slabs are shown on Standard Drawing BD-628M.

1.6 EXPANSION JOINTS

For total bridge lengths of 150 ft. or less (i.e. 75 ft. maximum expansion length), no provisions for expansion at the end of the approach slab shall be provided when the roadway has flexible pavement. For longer bridges when the roadway has flexible pavement and for all bridges when the roadway has concrete pavement, provisions shall be made for expansion, in the form of a strip seal expansion dam at the end of the approach slab. A short sleeper slab shall be provided beneath the expansion dam at the end of the approach slab and beginning of the rigid pavement. Details of the expansion joint (Details 13, 14 and 15) are shown on Standard Drawing BD-628M.

COMMENTARY

Wingwall reinforcement shown on Standard Drawing BD-667M was checked for the following cases:

1. Passive earth pressure acting on the wingwall
2. Collision force at Performance Level 2 (PL-2) acting near the free end of the wingwall plus active earth pressure.

The soil angle of internal friction used in designing the wingwall reinforcement is 30°.

C1.5

Providing a reinforced concrete approach slab tied to the bridge deck moves the expansion joint away from the end of the bridge. In addition, the approach slab eliminates settlement due to traffic compaction and backfill settling into the void left when the bridge contracts. It also prevents undermining of the abutments due to drainage at the bridge ends. Approach slabs are required despite the type of roadway pavement (rigid or flexible pavement).

The 25'-0" approach slab length is used in the analysis in all cases. The soil support under the approach slab was ignored to account for the possibility of soil settlement and erosion expected to take place directly behind the abutment.

Contraction joints at bridge abutments provide a controlled crack location rather than allowing a random crack pattern to develop.

The specified location of the dowels is intended to eliminate the development of negative moments in the approach slab along its connection to the abutment. This allows the approach slab to deflect without causing tension cracking at its top surface.

C1.6

Expansion joint at the end of the approach slab placed over a sleeper slab is a working joint. It opens and closes due to thermal expansion and contraction. The sleeper slab is provided to ensure proper movement of the structure and facilitate proper functioning of the expansion dam. It also provides protection against permanent migration.

SPECIFICATIONS

When a detached wingwall is used, provide a neoprene compression seal expansion joint between the abutment and the detached wingwall. Details of the expansion joint are shown on Standard Drawings BD-667M and BC-766M.

When possible, the expansion devices at the end of the approach slab and adjacent to detached wingwalls shall have a total range of movement equal to twice the abutment thermal movement, Δ_{max} , calculated using Ap.G1.2.7.4, but not less than 2 in.. The gap between the two sides of the expansion dam at the time of construction shall be such that it allows the expansion device to be subjected to a displacement Δ_{max} in expansion or in contraction safely.

In the case of a relatively large abutment design movements, expansion devices having a total range of movement twice the abutment thermal movement, Δ_{max} , may not be available. In these cases, the largest available expansion device of the type specified above for each location shall be used. The opening of the expansion dam at the time of construction shall be adjusted based on the expected total abutment movement, Δ_{total} , calculated using Ap.G1.2.7.4, and the actual construction temperature. The excess capacity of the expansion device beyond Δ_{total} shall be divided equally between expansion and contraction.

1.7 BEARING PADS

Plain, 50 durometer neoprene bearing pads shall be placed under all girders. The bearing pads shall be 3/4" thick and 1 ft. wide. The length of the bearing pads shall depend on the width of the bottom flange of the girders. The pads shall be placed as shown on Standard Drawing BD-653M.

Block the areas under the girders not in contact with the bearing pads using backer rods.

1.8 DESIGN DETAILS

Integral abutment standard details are shown on Standard Drawing BD-667M.

COMMENTARY

The calculations of the abutment movements are subject to a relatively high level of uncertainty due to the variation in soil properties and compaction conditions. The specified range of movements for expansion devices is intended to account for these uncertainties.

C1.7

Blocking the areas under the girders not in contact with the bearing pads is intended to prevent honeycombing of the surrounding concrete. Honeycombing will take place when the cement paste enters the 3/4" gap between the bottom of girders and the top of the pile cap in the areas under the girders not in contact with the bearing pads.

SPECIFICATIONS

COMMENTARY

1.9 REFERENCES

1. W. G. Fleming, A. J. Weltman, M. F. Randolph and W. K. Elson. Piling Engineering. Halsted Press, New York, 1985.
2. L. F. Greimann, R. E. Abendroth, D. E. Johnson and P. B. Ebner, “Pile Design and Tests for Integral Abutment Bridges”. Iowa Department of Transportation Report HR-273, 1987.
3. Richard M. Barker and Jay A. Puckett, Design of Highway Bridges, John Wiley and Sons, Inc., 1997.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX H – PENNSYLVANIA INSTALLATION DIRECT DESIGN (PAIDD) FOR CONCRETE PIPES

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1.0 SCOPE

- 1.1 Appendix H covers the direct design of buried precast concrete pipe using Standard Installations manufactured in accordance with Publication 408, Section 601, and the design and construction of the soil/pipe interaction system, intended for the conveyance of sewage, industrial wastes, storm water and drainage.
- 1.2 When buried, concrete pipe is part of a composite system comprised of the pipe and the surrounding soil envelope, which interact and contribute to the strength and structural behavior of the system.
- 1.3 Sections 6 through 14 of Appendix H present the PAIDD method for buried precast concrete pipe. This is a design and analysis method which accounts for the interaction between the pipe and soil envelope in determining loads, pressure distributions, moment, thrust and shear in the pipe, and which includes a procedure for calculating the required reinforcement.
- 1.4 Section 10 of Appendix H presents construction requirements for precast concrete pipe designed by the PAIDD method.
- 1.5 Appendix H shall be used by the designer working on Pennsylvania projects in preparing project specifications based on the PAIDD method.
- 1.6 The design procedures given in Appendix H are intended for use by engineers who are familiar with the installation and pipe characteristics that affect the structural behavior of buried concrete pipe installations and the significance of the installation requirements associated with each Standard Installation Type. Before applying the design procedures given in Sections 6 through 14, the engineer should review the guidance and requirements given in other sections of Appendix H and its reference documents.

2.0 APPLICABLE DOCUMENTS

2.1 American Society for Testing and Materials (Astm International)

- 2.1.1 C 822 Standard Terminology Relating to Concrete Pipe and Related Products
- 2.1.2 D 2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
- 2.1.3 D 2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
- 2.1.4 D 698 Standard Test Method for Laboratory Compaction Characteristics of Soil using Standard Effort (12,400 ft-lbf/ft³(600 kN-m/m³))
- 2.1.5 D 1557 Standard Test Method for Laboratory Compaction Characteristics of Soil using Modified Effort (56,000 ft-lbf/ft³(2700 kN-m/m³))

2.2 American Association of State Highway and Transportation Officials (AASHTO)

- 2.2.1 T 99 The Moisture-Density Relations of Soils using a 2.5 kg (5.5-lb) Rammer and a 305 mm (12-in.) Drop
- 2.2.2 T 180 The Moisture-Density Relations of Soils using a 4.54 kg (10-lb) Rammer and an 457 mm (18-in.) Drop

2.3 Pennsylvania Department of Transportation (PennDOT)

- 2.3.1 Design Manual, Part 4, Structures

3.0 DEFINITIONS

- 3.1 For definitions of terms relating to concrete pipe, see ASTM Definitions C 822.
- 3.2 For terminology related to soil classifications, see ASTM Classification D 2487 and ASTM Practice D 2488.

- 3.3 For terminology and definition of terms relating to structural design, see PennDOT Design Manual, Part 4 Structures.
- 3.4 Orientation Angle - An angular tolerance assumed for the position of the top of the pipe during the design of a pipe requiring a specific installation orientation because of the reinforcement cage configuration or the positioning of stirrups.
- 3.5 Prism Load - Weight of column of earth above the outside diameter of pipe.
- 3.6 Figure 3.6-1 illustrates the definitions and limits of the terms foundation, subgrade, bedding, haunch, lower side, sidefill, backfill or overfill, invert, crown, springline, top of pipe and bottom of pipe as used in this Standard Practice.

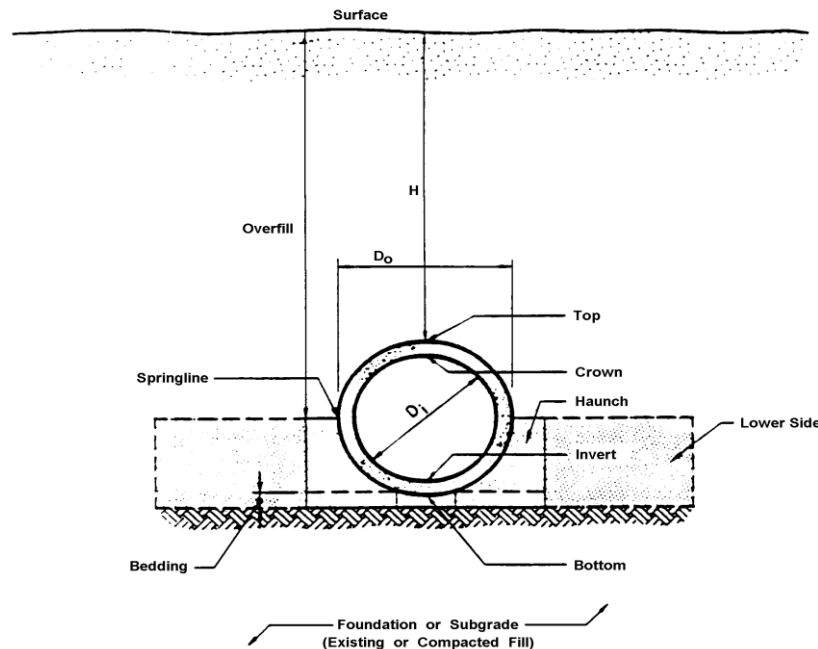


Figure 3.6-1 - Pipe/Installation Terminology

4.0 NOTATIONS

A_s	=	required tension reinforcement area (in. ² /ft.)
A_{si}	=	required total inner cage reinforcement area (in. ² /ft.)
A_{so}	=	required total outer cage reinforcement area (in. ² /ft.)
A_{vr}	=	required stirrup reinforcement area to resist radial tension forces (in. ² /ft.) in each line of stirrups at circumferential spacing, s_v
A_{vs}	=	required stirrup reinforcement area to resist shear (in. ² /ft.) in each line of stirrups at circumferential spacing, s_v
B_1	=	crack control coefficient for effect of spacing and number of layers of reinforcement
b	=	width of section which resists stress, 12 in./ft.
C_1	=	crack control coefficient for type of reinforcement
d	=	distance from compression face to centroid of tension reinforcement (in.)
D_i	=	inside diameter of pipe (in.)
D_o	=	outside diameter of pipe (in.)
f_s	=	maximum service load strength of reinforcing steel for crack control (ksi)
f_v	=	maximum allowable strength of stirrup material (ksi)

f_y	=	design yield strength of reinforcement (ksi)
F_c	=	factor for effect of curvature on diagonal tension (shear) strength in curved components
F_d	=	factor for crack depth effect resulting in increase in diagonal tension (shear) strength with decreasing d
F_{cr}	=	crack width control factor for adjusting crack control relative to an average maximum crack width of 0.01 in. at 1 in. from the tension reinforcement when $F_{cr} = 1.0$
F_{tp}	=	factor for process and materials that affect the radial tension strength of pipe
F_v	=	factor for crack depth effect resulting in increase in diagonal tension (shear) strength with decreasing d
F_{vp}	=	factor for process and materials that affect the shear strength of pipe
F_{rt}	=	factor for pipe size effect on radial tension strength
F_N	=	coefficient for effect of thrust on shear strength
f'_c	=	design compressive strength of concrete (ksi)
g_c	=	gravitational acceleration (ft./sec. ²)
h	=	overall thickness of member (wall thickness) (in.)
H	=	design height of earth above top of pipe (in.)
i	=	coefficient for effect of axial force at service load stress, f_r
l_0	=	total additional arc length beyond calculated arc lengths requiring stirrups (in.)
M_s	=	service load bending moment (kip-in./ft.)
M_u	=	factored moment acting on cross-section (kip-in./ft.)
M_{nu}	=	factored moment as modified for effects of compressive or tensile thrust (kip-in./ft.)
n	=	number of layers of reinforcement in a cage, 1 or 2
N_s	=	axial thrust acting on cross-section, service load condition (+ when compressive; - when tensile) (kip/ft.)
N_u	=	factored axial thrust acting on cross-section (+ when compressive, - when tensile) (kip/ft.)
PL	=	PL denotes the prism load (weight of the column of earth) over the pipe's outside diameter and is calculated as:

$$PL = \frac{wD_o}{12} [H + 0.01 D_o]$$

r	=	radius to centerline of pipe wall (in.)
r_s	=	radius of the inside reinforcement (in.)
s_v	=	circumferential spacing of stirrups (in.)
s_l	=	spacing of circumferential reinforcement (in.)
t_b	=	clear cover over reinforcement (in.)
V_b	=	basic shear strength of critical section, at $M_{nu}/(V_u d) = 3.0$
V_c	=	nominal shear strength provided by concrete cross-section (kip/ft.)
V_u	=	factored shear force acting on cross-section (kip/ft)
V_{uc}	=	factored shear force at critical section, where $M_{nu}/(V_u d) = 3.0$
w	=	unit weight of soil (kcf)
ρ	=	ratio of reinforcement area to concrete area
ϕ_f	=	strength reduction factor for flexure
ϕ_v	=	strength reduction factor for shear
ϕ_r	=	strength reduction factor for radial tension
θ	=	orientation angle (°)

5.0 SUMMARY OF PRACTICE

- 5.1 The PAIDD method is a direct design method that accounts for the interaction between the pipe and soil envelope in determining loads and distribution of earth pressure on a buried pipe.
- 5.2 The two Standard Installations, embankment and shoring/trench box, that are a part of this Standard Practice are based on the results of research on pipe/soil interaction, together with evaluation of current construction practice, equipment, procedures and experience.
- 5.3 Earth load effects are determined from the pressure distribution coefficients for the two Standard Installations (see Standard Drawing BD-636M, Sheet 1 for Soil Pressure Distribution Model and Sheet 2 for Tables B and C).

5.4 The structural design of concrete pipe is based on a limits state design procedure that accounts for strength and serviceability criteria. The design criteria include: structural aspects, such as flexure, thrust, shear and radial tension strengths; handling and installation; and crack width control.

5.5 The design of a concrete pipe for these particular Standard Installations is based on the assumption that the specified design bedding and fill requirements will be achieved during construction of the installation.

6.0 GENERAL

6.1 Design procedures and criteria shall conform to applicable sections of this Standard Practice.

6.2 The manufacturer shall submit to the owner for approval a detailed design of the pipe based on the Owner's Design Requirements.

7.0 DESIGN REQUIREMENTS

7.1 The designer shall establish the following criteria and requirements:

7.1.1 Intended use of pipeline.

7.1.2 Pipe inside diameter, D_i .

7.1.3 Pipeline plan and profile drawings with installation cross-sections as required.

7.1.4 Design earth cover height above the top of the pipe, H (see Figure 3.6-1).

7.1.5 Allowable Standard Installation Types. Embankment or shoring/trench box installations.

7.1.6 Performance requirements for pipe joints, for sanitary installation.

7.1.7 Design live and surcharge loading, if heavier than standard PennDOT design loads.

7.1.8 Design intermittent internal hydrostatic pressures, if required.

7.1.9 Crack width control criteria, other than specified in Section 7.2, if permitted by the Chief Bridge Engineer.

7.2 The following design requirements shall apply:

7.2.1 Load Factors

Dead and Earth Load Factor (Shear and Moment)	1.3

Dead and Earth Load Factor (Thrust)	
Reinforcement Design	1.0
Concrete Compression	1.3

Live Load Factor (Shear and Moment)	2.17
Impact Factors	$IM = 40 (1.0-0.125H)$

7.2.2 Strength Reduction (ϕ) Factors

	<u>Heavy Duty</u>	<u>Standard Duty</u>
Flexure	$\phi_f = 0.9$	$\phi_f = 0.95$
Radial Tension	$\phi_v = 0.85$	$\phi_v = 0.9$
Diagonal Tension	$\phi_r = 0.85$	$\phi_r = 0.9$

- 7.2.3** Crack Control Factor 0.7
(unless modified by the owner)
- 7.2.4** Orientation Angle $\theta = \pm 10^\circ$
- 7.2.5** Process and Material Factors
- | | |
|------------------|-----|
| Radial Tension | 1.0 |
| Diagonal Tension | 1.0 |
- 7.3** The manufacturer shall submit the following manufacturing design data to the owner for approval.
- 7.3.1** Pipe Wall Thickness
- 7.3.2** Concrete Strength
- 7.3.3** Reinforcement specification, reinforcement Type 2 or 3 as shown in Table 12.4.1-1, design yield strength, placement and design concrete cover, cross-sectional diameters, spacing, cross-sectional area, description of longitudinal members, and if used, developable stirrup design stress, stirrup shape, placement and anchorage details.
- 7.3.3.1** Crack Control Coefficient, C_1 , as indicated in Table 12.4.1-1. Note that Standard Drawing BD-636M and PAIDD software have values built-in for Type 2 reinforcement only.
- 7.3.3.2** The minimum design concrete cover over the reinforcement shall be 1 in. in pipe having a wall thickness of 2 1/2 in. or greater, and 3/4 in. in pipe having a wall thickness of less than 2 1/2 in.. For stirrup design provide 1 1/4 in. of concrete cover to the circumferential reinforcement.
- 7.3.4** Pipe laying length and joint information.
- 7.3.5** The yield strength and ultimate strength of the tension reinforcement used for design shall be as specified in 8.2.1 or 8.2.2.

8.0 MATERIALS

8.1 Concrete

- 8.1.1** Concrete shall conform to the requirements of Publication 408, Section 601.

8.2 Reinforcement

- 8.2.1** Reinforcement shall consist of cold-drawn steel wire conforming to ASTM Specification A 82, or ASTM Specification A 496, or of cold-drawn steel welded wire fabric conforming to ASTM Specification A 185, or ASTM Specification A 497, or of hot-rolled steel bars conforming to ASTM Specification A 615.
- 8.2.2** The use of cold drawn steel or cold drawn steel welded wire fabric with strengths exceeding ASTM Specification values may be approved by the owner when the reinforcing manufacturer's mill test report certifies a higher minimum yield and ultimate strength steel is being provided. The other requirements of the appropriate ASTM Specifications listed in 8.2.1 (A 82, A 496, A 185 or A 497) shall be met by the higher minimum strength steels. The yield strengths shall not be taken greater than 86% of the ultimate strength, or 80 ksi, whichever is lower. Note that Standard Drawing BD-636M and PAIDD use 65 ksi yield strength.
- 8.2.2.1** Section 8.2.2 does not apply to wire sizes having a nominal diameter of less than 0.088 in. or nominal cross-sectional area of less than 0.005 in². Section 8.2.2 does not apply to (any size of) hot-rolled steel manufactured in accordance with ASTM A 615.

9.0 LOADS

9.1 Dead Loads

- 9.1.1** The dead load of the pipe weight shall be considered in the design and based on a reinforced concrete density of 0.150 kcf, unless otherwise specified.
- 9.1.2** The earth load from the fill over the pipe shall be based on the design soil unit density of 0.140 kcf.
- 9.1.3** For unpaved and flexible pavement areas, the minimum fill, including flexible pavement thickness, over top outside of the pipe shall be 1 ft., or 1/8 of the inside diameter, whichever is greater. Under rigid pavements, the distance between the top of the pipe and the bottom of the pavement base course shall be a minimum of 1 ft. of compacted granular fill. However, to avoid conflict with other design and field constraints refer to Standard Drawing RC-30M for vertically locating the pipe considering minimum cover requirements specified in the standard. Pipe is not to be placed in the pavement structure without Chief Bridge Engineer approval.
- 9.1.4** The dead load of fluid in the pipe shall be based on a unit density of 0.0624 kcf, unless otherwise specified.

9.2 Live Loads

- 9.2.1** Truck loads shall be the AASHTO HS25. The impact factor shall be:
- $$IM = 40(1.0 - 0.125H) \quad (9.2.1-1)$$
- 9.2.2** Railroad loads shall be the AREMA designated Cooper E-series.
- 9.2.3** Aircraft or other live loads shall be as specified by the owner.

9.3 Intermittent Internal Hydrostatic Pressure

- 9.3.1** Internal hydrostatic pressure caused by hydraulic surcharges or other temporary hydraulic conditions shall be as specified by the owner.

10.0 STANDARD INSTALLATIONS

- 10.1** Soil classifications are in accordance with ASTM Classification D 2487 and Practice D 2488. For step-by-step installation refer to Standard Drawing RC-30M.
- 10.2** The soil-type and compaction requirements for the Standard Installations are presented in Standard Drawings RC-30M and BD-636M. Table 10.2-1 relates the Standard Installations designated soils to the AASHTO and Unified Soil System Classifications categories.

Table 10.2-1 - Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations

SIDD Soil	Representative Soil Types		Percent Compaction	
	USCS	AASHTO	Standard Proctor	Modified Proctor
Gravelly Sand (SW)	SW, SP GW, GP	A1, A3	100	95
			95	90
			90	85
			85	80
			80	75
			61	59
Sandy Silt (ML)	GM, SM, ML Also GC, SC with less than 20% passing #200 sieve	A2, A4	100	95
			95	90
			90	85
			85	80
			80	75
			49	46
Silty Clay (CL)	CL, MH, GC, SC	A5, A6	100	90
			95	85
			90	80
			85	75
			80	70
			45	40
	CH	A7	100	90
			95	85
			90	80
			45	40

10.3 The soil types and compaction requirements for the Standard Installations are defined as follows:

10.3.1 Soil Materials

SW, Well-graded sands, gravelly sands, little or no fines (also includes GW - well graded gravels, gravel-sand mixtures little or no fines, and crushed stone).

ML, Inorganic silts, fine sands or clayey silts with slight plasticity. (May be used for native granular soils.)

CL, Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.

10.3.2 Compaction Specifications

SP, Standard Proctor Density - AASHTO T 99 or ASTM D 698

MP, Modified Proctor Density - AASHTO T 180 or ASTM D 1557

L, Loose - Uncompacted

10.3.3 In-Situ Materials (INS)

R - Rock
F - Firm to hard
M - Medium
A - Average
S - Soft
SCR - Loosened by scarifying

11.0 PRESSURE DISTRIBUTION

- 11.1** The pressure distributions on the pipe from applied loads and bedding reaction as determined from soil-structure analyses for the Standard Installations are presented in Standard Drawing BD-636M. Also listed are the relative dimensions and coefficients, based on a unit prism load on the pipe. The use of the computer program "PAIDD" simplifies the analysis of a pipe for the moments, thrusts and shears caused by these pressure distributions.

12.0 REINFORCEMENT

12.1 Flexural Strength

- 12.1.1** Reinforcement for Flexural Strength shall be not less than A_s , where:

$$A_s f_y = g \phi_f d - N_u - \sqrt{g [g (\phi_f d)^2 - N_u (2 \phi_f d - h) - 2 M_u]} \quad (12.1.1-1)$$

$$g = b f'_c [0.85 - 0.05 (f'_c - 4)] \quad (12.1.1-2)$$

$$g_{\max} = 0.85 b f'_c$$

$$g_{\min} = 0.65 b f'_c$$

$$b = 12 \text{ in./ft.}$$

12.2 Minimum Reinforcement

- 12.2.1** The following equation applies only to circular pipe larger than 33 in. in diameter with circular reinforcement. For inside face of pipe, reinforcement shall not be less than A_{si} , where:

$$A_{si} = 1.1 \frac{(D_i + h)^2}{1000 f_y} \quad (12.2.1-1)$$

to account for transportation stresses.

- 12.2.2** For outside face of pipe, reinforcement shall not be less than A_{so} , where:

$$A_{so} = 0.65 \frac{(D_i + h)^2}{1000 f_y} \quad (12.2.2-1)$$

- 12.2.3** For pipe 33 in. diameter and smaller, with a single cage reinforcement in the middle third of the pipe wall, reinforcement shall not be less than A_s , where:

$$A_s = 2.2 \frac{(D_i + h)^2}{1,000 f_y} \quad (12.2.3-1)$$

12.3 Maximum Flexural Reinforcement without Stirrups or Ties

- 12.3.1** When stirrups are not used, the wall shall be designed so that the flexural reinforcement required by 12.1 does not exceed " $A_s \text{ max}$ " as determined by 12.3.2 or 12.3.3.

12.3.2 Limited by Radial Tension

$$A_{s \max} f_y = 0.506 r_s F_{rp} \sqrt{f'_c} \left(\frac{\phi_r}{\phi_f} \right) F_{rt} \quad (12.3.2-1)$$

$$F_{rt} = 1 + 0.00833 (72 - D_i) \text{ for } 12 \text{ in.} < D_i \leq 72 \text{ in.} \quad (12.3.2-2)$$

$$F_{rt} = \frac{(144 - D_i)^2}{26,000} + 0.80 \text{ for } 72 \text{ in.} < D_i \leq 144 \text{ in.} \quad (12.3.2-3)$$

$$F_{rt} = 0.8 \text{ for } D_i > 144 \text{ in.} \quad (12.3.2-4)$$

12.3.3 Limited by Concrete Compression

$$A_{s \max} f_y = \left[\frac{55 g' \phi_f d}{87 + f_y} \right] - 0.75 N_u \quad (12.3.3-1)$$

$$g' = b f'_c \left[0.85 - 0.05 \frac{(f'_c - 4)}{1} \right] \quad (12.3.3-2)$$

$$g'_{\max} = 0.85 b f'_c$$

$$g'_{\min} = 0.65 b f'_c$$

$$b = 12 \text{ in./ft.}$$

12.4 Crack Control

12.4.1 Crack control is assumed to be at 1 in. from the tension reinforcement. The crack control factor, F_{cr} , shall not exceed that specified, where:

$$F_{cr} = \frac{B_1}{30,000 \phi_f d A_s} \left[\frac{M_s + N_s \left(d - \frac{h}{2} \right)}{ij} - 0.0316 C_1 b h^2 \sqrt{f'_c} \right] \quad (12.4.1-1)$$

C_1 – See Table 12.4.1-1

$$b = 12 \text{ in./ft.}$$

$$j = 0.74 + 0.1 \frac{e}{d} \quad (12.4.1-2)$$

$$j_{\max} = 0.9$$

$$i = \frac{1}{1 - \frac{jd}{e}} \quad (12.4.1-3)$$

$$e = \frac{M_s}{N_s} + d - \frac{h}{2}, \text{ in.} \quad (12.4.1-4)$$

if $e/d_{\min} < 1.15$ crack control will not govern

$$B_1 = \sqrt[3]{\frac{t_b s_l}{2n}} \quad (12.4.1-5)$$

s_l = spacing of circumferential reinforcement, in.

n = 1, when tension reinforcement is a single layer

n = 2, when tension reinforcement is made of multiple layers

Table 12.4.1-1 – Crack Control Coefficients

Type of Reinforcement	C_1
Smooth wire or plain bars	1.0
Welded smooth wire fabric with 8 in. maximum spacing of longitudinals, welded deformed wire fabric, or deformed wire	1.5
Deformed bars or any reinforcement with stirrups anchored thereto	1.9

12.4.2 If the service load thrust, N_s is tensile rather than compressive (this may occur in pipes subject to intermittent hydrostatic pressure), the quantity $(1.1 M_s - 0.6 N_s d)$ shall be used (with tensile N_s taken negative) in place of the quantity $[M_s + N_s (d - h/2)]/ij$ in Equation 12.4.1-1.

12.5 Shear Strength

12.5.1 The area of reinforcement, A_s , determined in Section 12.1 and 12.4, shall be checked for shear strength adequacy, so that the basic shear strength, V_b , is greater than the factored shear force, V_{uc} , at the critical section located where $M_{nu}/V_{ud} = 3.0$. See Equation 12.5.1-7 for M_{nu} .

$$V_b = 0.0316 b \phi_v d F_{vp} \sqrt{f'_c} (1.1 + 63\rho) \left[\frac{F_d F_N}{F_c} \right] \quad (12.5.1-1)$$

$b = 12 \text{ in./ft.}$

$f'_{c \text{ max}} = 7 \text{ ksi}$

$$\rho = \frac{A_s}{bd}$$

$$\rho_{\max} = 0.02 \quad (12.5.1-2)$$

$$F_d = 0.8 + \frac{1.6}{d} \quad (12.5.1-3)$$

max $F_d = 1.3$, for pipe with two cages

max $F_d = 1.4$, for pipe through 36 in. diameter with a single circular cage

$$F_c = 1 \pm \frac{d}{2r} \quad (12.5.1-4)$$

(+) tension on the inside of the pipe

(-) tension on the outside of the pipe

For compressive thrust (+N_u):

$$F_N = 1 + \frac{N_u}{24h} \quad (12.5.1-5)$$

For tensile thrust (-N_u):

$$F_N = 1 + \frac{N_u}{6h} \quad (12.5.1-6)$$

$$M_{nu} = M_u - N_u \frac{(4h - d)}{8} \quad (12.5.1-7)$$

12.5.2 If V_b is less than V_{uc} stirrups shall be provided. See Section 12.6.

12.6 Stirrups

12.6.1 If stirrups are required for radial tension by 12.3.2 or for shear by 12.5, they shall meet the following requirements.

12.6.2 Radial Tension Stirrups

$$A_{vr} = \frac{1.1s_v (M_u - 0.45 N_u \phi_r d)}{f_v r_s \phi_r d} \quad (12.6.2-1)$$

f_{v max} = f_y or anchorage strength, whichever is less

$$s_{v \max} = 0.75\phi_r d \quad (12.6.2-2)$$

12.6.3 Shear Stirrups

$$A_{vs} = \frac{1.1s_v}{f_v \phi_v d} [V_u F_c - V_c] + A_{vr} \quad (12.6.3-1)$$

$$V_c = \frac{4V_b}{\frac{M_{nu}}{V_u d} + 1} \quad (12.6.3-2)$$

$$V_{c \max} = 0.0632\phi_v b d \sqrt{f'_c} \quad (12.6.3-3)$$

b = 12 in./ft.

12.6.4 Extent of Stirrups

12.6.4.1 When stirrups are required at the invert or crown regions for shear strength, or for shear and radial tension, they shall be spaced at s_v along the inner reinforcing and extend over a basic length on each side of the invert or crown where V_u is greater than V_c, plus an additional minimum arc length 0.5 l₀ from each end of the basic arc length to allow for installation variations up to the orientation angle, θ, where

$$l_{\theta} = \frac{\pi\theta}{180} (D_i + 2t_b) + h \quad (12.6.4.1-1)$$

If stirrups are also required at the springline region (may occur in very high loading conditions), they shall be spaced at s_v and shall extend around the entire pipe circumference.

12.6.4.2 When stirrups are required for radial tension only, they shall be spaced at s_v along the inner reinforcing and extend over a basic length to the locations on each side of the invert or crown where the required $A_s f_y$ is less than the limiting value given by Equation 12.1.1-1, plus an additional minimum arc length $0.5 l_{\theta}$ from each end of the basic arc length to allow for installation variations up to the orientation angle, θ .

12.6.5 Stirrup Anchorage

12.6.5.1 Anchorage of both ends of the stirrup shall be sufficient to develop the factored stress in the stirrup. The maximum factored tensile stress in the stirrup shall be the yield strength or the stress that can be developed by anchorage shall be considered to be 65 ksi for continuous stirrups, prefabricated stirrup mats and bent wire shear steel as shown in PAIDD Manufacturing Standard (Publication 280).

12.6.5.2 Anchorage systems that differ from those shown in PAIDD Manufacturing Standard (Publication 280) and corresponding anchorage design stresses may be submitted to the owner for approval.

12.7 Laps, Splices and Development of Reinforcement

13.0 PIPE DESIGNATION

13.1 The pipe designation shall be in accordance with Publication 408, Section 601.

14.0 MULTIPLE PIPE INSTALLATIONS

Multiple pipe installations are typically used at locations where limited cover prevents the use of a single pipe of larger cross-sectional area. A minimum of 2 ft. clearance between adjoining pipes shall be provided. See Figures 14.0-1 and 14.0-2.

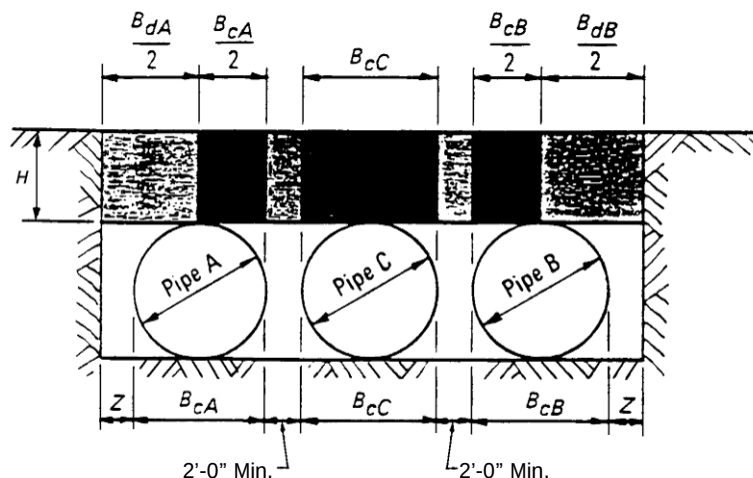


Figure 14.0-1 - Flat Trench Multiple Pipe Installation

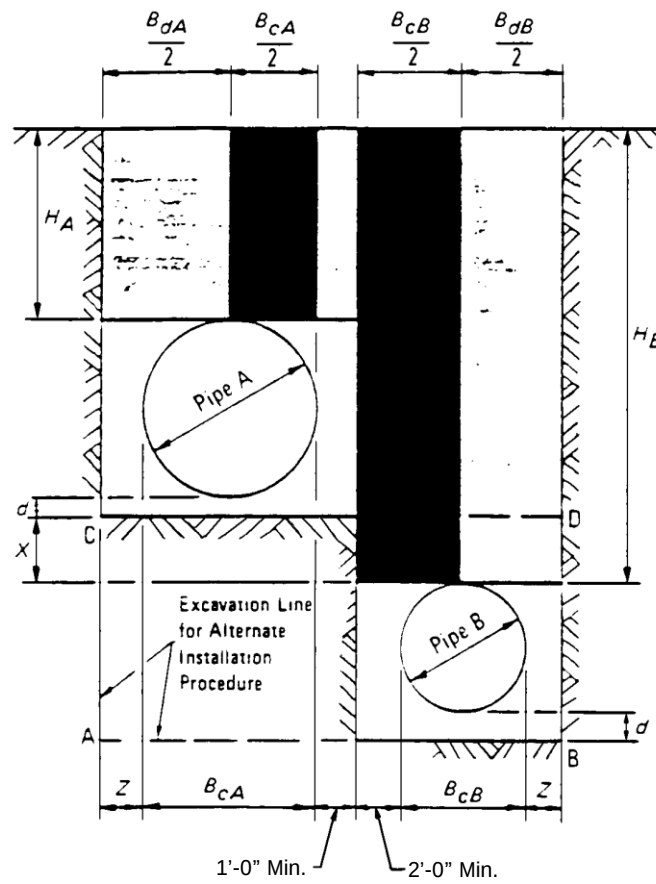


Figure 14.0-2 - Benched Trench Installation

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
DESIGN MANUAL

PART 4

VOLUME 2

APPENDIX I – EXTENSIBLE REINFORCEMENTS

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1.1 EXTENSIBLE REINFORCEMENT

Extensible reinforcement may be used for MSE walls if their performance has been substantiated either by tests or experience and approval by the Department. Reinforcements may be manufactured from long chain polymers, such as polypropylene, HDPE and polyester. The design of MSE walls with extensible reinforcements shall comply with provisions of A11.10 and D11.10, except as modified in this Appendix.

COMMENTARY

C1.1

The most commonly used extensible reinforcement materials are geotextiles and geogrids made of polyethylene, polyester and polypropylene. The three main types of geotextiles are wovens, nonwovens and knitted. Only woven and nonwoven geotextiles are used for soil reinforcement applications. There are two main types of geogrids, those manufactured by drawing a perforated polymer sheet in one or two perpendicular directions and those manufactured by overlapping perpendicular or nearly perpendicular polymer strands, which are then bonded at their junctions.

Extensible reinforcements are characterized by reinforcement tensile moduli that are relatively low and result in working reinforcement strains greater than those associated with full mobilization of active lateral earth pressures. This property significantly affects the state of stress in the reinforced zone and the location of the internal failure zone and determines the outward deflection of the structure. Extensible reinforcement tensile moduli are significantly lower by one or two orders of magnitude than the tensile moduli of inextensible reinforcements such as steel.

Polymers at present do not have sufficient in-ground use to predict behavior based on observed performance. Experience based on over 20 years with plastic pipe performance strongly suggests that time, temperature, mechanical damage, molecular structure, as affected by radiation (UV) and chemical exposure, microbiological attack, environmental stress cracking hydrolysis, plasticization and synergistic effects from all these variables can significantly affect stress strain behavior and durability for in-ground service.

To enhance durability, additives are incorporated in the manufacturing process to prevent high temperature degradation during the extrusion process: to protect against UV attack by the addition of finely divided Carbon Black which screens the polymer or hindered amine light stabilizers (HALS) which function by interrupting the radical chain degradation mechanism, and with the inclusion of long-term anti-oxidants to further enhance the inherent resistance of polymers to the aging process. The degree of protection or total resistance achieved is based on the selection of proper additives which will vary from product-to-product.

To protect against UV exposure polymers generally contain for maximum protection 2.0% to 3.0% of finely divided carbon black well dispersed in the polymer. Alternately, the addition of hindered amine light stabilizers (HALS) has been shown to be effective in providing UV resistance.

Environmental Stress Cracking (ESC) is the process under which premature failure can occur without chemical change in polymers subjected to concurrent stress in a particular environment. Uniaxially stretched geogrids may be prone to environmental stress cracking in the rib portion which is generally non-oriented.

SPECIFICATIONS

COMMENTARY

1.1.1 Technical Considerations, Restrictions

Extensible reinforcement may be used in lieu of steel inextensible reinforcements. It is particularly well suited where the soil environment is aggressive and corrosion is anticipated or where stray current sources are present, or where the fill is expected to settle significantly different in both the longitudinal or transverse directions.

Restrictions outlined in A11.10.1 and D11.10.1 shall apply, except as follows:

- (a) The impervious membrane, for walls supporting roadways, may be omitted if directed by the District Bridge Engineer.
- (b) Walls shall be limited to 35 ft. heights.

1.1.2 Structure Dimensions

Requirements for structure dimensioning, embedment depth, bearing capacity of foundations, external stability considerations and drainage shall be as per the applicable sections of A11.10 and D11.10.

An additional requirement is the determination of the factor of safety against sliding of the reinforced backfill directly on a layer of reinforcement. For this calculation, the soil reinforcement interface friction shall be determined.

For geotextiles or geogrids, the angle of friction associated with direct sliding of the reinforced backfill on the geotextile or geogrid, ρ , shall be obtained from direct shear tests carried out in accordance with ASTM D 5321

For polyester fibers in certain forms, degradation has been reported as a result of hydrolysis generated by absorption of water by the fibers in alkaline environments, by thermal degradation or due to UV exposure. For maximum resistance to strength losses due to hydrolysis, polyesters should be formulated to high molecular weights ($>25,000$) and with low Carboxyl End Group numbers ($\text{CEG} < 25$) and should not be used in highly acidic or alkaline environments (i.e., if the pH is less than 3.0 or more than 9.0).

Polypropylenes are only slightly affected by hydrolysis as their absorption characteristics of water are considerably smaller than those of polyesters, but lose strength due to oxidation, especially in the presence of transition metals (Fe , Cu , Mn , etc. compounds) in the backfill.

The inclusion of effective primary, secondary and synergistic antioxidants are essential in extending the lifetimes of all polyoleofin products. New anti-oxidants are under development to extend survivability. Other aging losses have not been systematically evaluated.

Significant research is presently underway to quantify potential strength losses as a function of polymer characteristics and environment.

C1.1.1

MSE walls with extensible reinforcements are not generally economical at greater heights.

C1.1.2

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for Soil-Geosynthetic Friction.

In the absence of specific test data, the angle of soil reinforcement friction, ρ , shall be taken as 2/3 of the internal angle of friction of the backfill used in the reinforced zone.

1.1.3 Loading

The requirements of A11.10.3 shall apply.

1.1.4 Internal Stability

Internal stability for structures constructed with polymeric reinforcements shall be analyzed using a tie-back wedge method approach.

It is assumed that the full shear strength of the reinforced fill is mobilized and active lateral earth pressures are developed. The assumed failure plane is defined by the Rankine active earth pressure zone defined by a straight line passing through the wall toe and oriented at an angle of $45^\circ + \phi/2$ from the horizontal for both horizontal and sloping backfill conditions, and where ϕ is the effective friction angle of the soil in the reinforced zone.

The tensile force in the reinforcement is a function of the vertical stress due to self weight, uniform normal surcharge and active thrust multiplied by K_a . Reinforcement tensions induced by vertical or horizontal line loads, or by point loads shall be added by superposition to the tensile forces induced by the reinforced wall fill soil and the retained backfill. The method of computation shall assume an unyielding rigid wall rotating about its toe.

The value of K_a in the reinforced soil mass is assumed to be independent of all external loads, except sloping fills. The maximum friction angle used for the computation of horizontal stress within the reinforced soil mass, composed of select backfill, shall be 34° in the absence of backfill specific tests. Where site-specific tests are performed, the soil strength shall be evaluated at residual stress levels.

Minimum grid or geotextile length in the resistant zone shall be 3 ft.. The total reinforcement length at any level shall be equal to the sum of the lengths in the active and resistant zone, but shall be in no case less than 8 ft..

1.1.5 Movements Under Service Limit State

The provisions of A11.10.4 shall apply.

Horizontal wall movements may approach $H/75$, mainly during construction or roughly three times the movements anticipated with extensible reinforcement.

1.1.6 Safety Against Soil Failure

The provisions of A11.10.5 and D11.10.5 shall apply.

COMMENTARY

FHWA DP82-1 provides further details of the use of ρ taken as 2/3 of the internal angle of friction of the backfill.

C1.1.4

The tie-back wedge procedure is a limit equilibrium method of design that assumes full mobilization of the reinforced fill shear resistance and generation of active lateral earth pressures. It is assumed that the full shear strength of the reinforced fill is mobilized so that the soil mass bounded by the failure plane moves as a rigid body and active lateral earth pressures are developed. Consequently, Rankine theory is applicable to the design of MSE walls with extensible reinforcements. Applicability of Rankine theory to the design of MSE walls with extensible, i.e. polymeric, reinforcements has been documented by Bonaparte, et al, (1986) and Christopher, et al, (1990).

The residual stress level is determined as the stress after failure at which strain continues to increase with no significant change in stress.

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1.1.7 Safety Against Structural Failure

Polymeric materials exhibit creep (time and temperature dependent) behavior. Long-term stress-strain-time behavior of the reinforcement shall be determined from results of controlled laboratory creep tests conducted for a minimum duration of 10,000 hours for a range of load levels on samples of the finished product in accordance with ASTM D 5262. Samples shall be tested in the direction in which the load will be applied in use in either a confined or unconfined mode. Results shall be extrapolated to the required design life using procedures outlined in ASTM D 2837.

From these tests, the following shall be determined:

- The highest load level at which the log time creep-strain rate continues to decrease with time within the required lifetime and no failure either brittle or ductile can occur. This value shall be termed the Limit State Tensile Load, designated as T_p .
- The tension level at which total strain is not expected to exceed 5% within the design lifetime. This value of load shall be designated T_s , the Serviceability State Tensile Load.

The effects of aging, chemical and biological exposure, environmental stress cracking, stress relaxation, hydrolysis and variations in the manufacturing process, as well as the effects of construction damage shall be evaluated and extrapolated to the required design life.

For these evaluations, the specified design life shall be a minimum of 75 years and the assumed in ground temperature shall be 70° F. Permanent structures shall have a design life of 100 years.

1.1.8 Resistance Factors

The following resistance factors shall be evaluated when using extensible polymeric reinforcements. For strength and extreme event load conditions, the limit state tensile load T_p is applicable. For service load conditions, the serviceability tensile load T_s is applicable.

COMMENTARY

C1.1.7

The methodology for determining tensile strength has been adopted from the recommended practice proposed by Task Force 27 of AASHTO-AGC-ARTBA.

Where polymeric reinforcements are expected to permanently resist tensile loads, creep derived tensile strength will govern the determination of useable tensile stresses.

In lieu of full-scale construction damage tests and substantial experimental data regarding the long-term effects of environmental conditions and aging, default resistance factors outlined in C1.1.8 should be used.

C1.1.8

The resistance factors for tensile strength of polymeric reinforcements must reflect creep reduction factors as well as reduction factors for aging and environmental losses and construction damage.

Creep derived limiting stresses at the expected service temperatures and projected life of the structure are obtained from laboratory creep tests conducted for minimum durations of 10,000 hours in general accordance with procedures outlined by McGowan (1986) or ASTM D 5262, which are modifications to creep test procedures (ASTM D 2837) commonly used to determine pressure ratings on thermoplastic pipe.

In the absence of such data, NCHRP 290 (Mitchell and Villet, 1987), and the FHWA Geotextile Engineering Manual (Christopher and Holtz, 1985) suggest certain reduction factors to the tensile resistance obtained from ASTM D 4595 for various types of polymers to determine a creep-derived limiting tensile resistance.

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<u>Polymer Type</u>	<u>Creep Reduction Coefficients</u>
Polyester	0.40
Polypropylene	0.20
Polyethylene	0.20

The effects of aging, oxidation, hydrolysis, radiation and chemical effects are considered in a additional resistance factor for each polymer type. At present, very little data is available from the manufacturers to determine quantitatively the effects of these mechanisms for a 75-year minimum design life. Therefore, default resistance factors should be adopted to the creep-derived limit state load, unless substantial experimental data for each specific polymer is developed by the manufacturer and presented. Short-term EPA 9090 test data is not considered sufficient to quantify an appropriate long-term strength reduction factor. For polyoleofin products (PP, HDPE) oxidation strength losses may be determined using oven aging tests in accordance with methods outlined by Wisse (1990). For polyester products, hydrolysis strength losses may be determined in accordance with methods outlined by McMahon (1959). In the absence of product specific data, a resistance factor for all aging effects, FD shall be taken as 2.0. In no case shall FD be less than 1.1.

The construction damage reduced resistance factor shall be determined from the results of full-scale construction damage tests using representative fill materials and construction procedures. In the absence of product-specific construction damage test data, the resistance factor for construction damage, FC, shall be taken as 3.0. For fills specified in Publication 408, this reduction factor can vary from 1.05 to 1.4 for sand fills containing no gravels and from 1.4 to 3.5 for fills containing significant gravel percentages. The lower values are generally associated with geogrids and the higher values with lightweight geotextiles.

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Table 1.1.8 – Resistance Factors for Extensible Reinforcement

CONDITION	TENSILE LOAD EVALUATION FACTORS	RESISTANCE FACTORS
Strength, Extreme Events	T_p from laboratory creep tests of 10,000 hours minimum duration FC and FD for each specific product and backfill	$0.8 \left[\frac{1}{FC \times FD} \right]$
Service	T_5 from laboratory creep tests of 10,000 hours minimum duration at a maximum strain of 5% FC and FD for each specific product and backfill	$0.8 \left[\frac{1}{FC \times FD} \right]$

1.1.9 Pullout Design Parameters

The applicable provisions for extensible polymeric reinforcements in A11.10.6.3.2 and D11.10.6.3.2 shall apply. The coefficient f_d , an interaction coefficient, for geogrids shall be determined using ASTM D 6706. It shall be evaluated under both short-term and long-term conditions. In the latter case, the pullout load shall be increased systematically up to 1.5 times the geogrid design load and kept constant for 1000 hours. The pullout load shall then be increased until failure occurs. Only the long-term coefficient shall be used for design. The test shall be performed using site-specific soil. The ultimate pullout load shall be based on a maximum elongation of the embedded geogrid of 3/4 in. as measured at the leading edge of the compressive zone and not the ultimate pullout capacity.

1.1.10 Connection Design

Extensible reinforcement connections to the facing shall be designed to carry the maximum factored load at each reinforcement level. A representative connection shall be load tested in order to determine the allowable connection working load.

Polymeric reinforcement strength is temperature dependent, generally decreasing at higher temperatures. Resistance factors derived under 1.1.8 are for in-ground service assumed at 70° F. Connection service temperatures can be considerably higher and, therefore, for creep reduction and aging, relevant higher service temperatures shall be determined.

C1.1.10

Service temperatures at connections have been measured to be nearly equal to ambient maximum temperatures.

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1.1.11 Construction Specifications

Construction specifications (Special Provisions) for MSE walls with concrete panels and polymeric reinforcements shall be the same as for Mechanically Stabilized Earth walls, with the following exceptions:

1. Section I. Description - Replace "strip or grid type metallic reinforcements" with "sheet or grid type polymeric reinforcements" in the description text.
2. Section III. Materials - Replace Parts 1, 2 and 3 of "(b) Reinforcement" with the following:
 - (1) Polymeric Grid Reinforcement - Provide a polymeric geogrid with a junction strength greater than as specified hereinafter. The summation of the shear strengths of joints occurring in a 12 in. length of grid sample shall be greater than the ultimate tensile strength of the grid element to which they are attached. The ultimate tensile strength shall be determined from wide width tensile tests (ASTM D 4595). Junction strength shall be measured in accordance with GRI Method, G.G. 2-87. The grid shall have been prequalified by the engineer for in-ground reinforcement use, with design values established for long-term resistance factors based on creep, aging and construction damage testing as shown on the plans.
 - (2) Polymeric Sheet Reinforcement - Provide a polymeric sheet reinforcement manufactured from virgin polymers meeting the requirements of Publication 408, Section 735, Class 4. The sheet reinforcement shall have been prequalified by the engineer for in-ground reinforcement use with design values established for long-term resistance factors based on creep, aging and construction damage testing as shown on the plans.
 - (3) Fasteners - Provide 1/2 in. (nominal)-diameter PVC pipe, Schedule 80, or approved equal for pinned geogrid connection when used.
3. Section III. Materials - The electrochemical criteria with respect to resistivity, chlorides and sulfates in Part 4 of "(e) Granular Fill Material" are waived.
4. Section IV. Construction - Add the following after the second paragraph of "(e) Backfilling".

Place the geogrid or geotextile flat on the compacted material and attach it to the facing according to the plans. Pretension the reinforcement such that it is

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free of wrinkles or any slack at the connection with the facing. Maintain the pretensioning until sufficient backfill is placed to keep the reinforcement secure. Cover the reinforcement with a minimum lift of 6 in. before compaction equipment is allowed to operate. Avoid the development of wrinkles and deformations of the reinforcement during the placement and compaction of backfill material.

2.0 REFERENCES

Bonaparte, R., Holts, R. D. and Giroud, J. P., "Soil Reinforcement Design Using Geotextiles and Geogrids", GEOTEXTILE TESTING AND THE DESIGN ENGINEER, ASTM STP 952, J. E. Fluett, Jr., ed., Philadelphia, Pennsylvania, pp. 69-116, 1986

Christopher, B. R., Gill, S. A., Giroud, J., Juran, I., Mitchell, J. K., Schlosser, F. and Dunncliff, J., "Reinforced Soil Structures, Volume I, Design and Construction Guidelines", FHWA RD-89-043, Federal Highway Administration, U. S. Department of Transportation, U. S. Government Printing Office, Washington, D. C., 301p., 1990

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**APPENDIX J i APPROVED COMMERCIALLY AVAILABLE OR
CONSULTANT - DEVELOPED SOFTWARE**

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1.0 COMPUTER PROGRAM ACCEPTANCE - GIRDER BRIDGES

1.1 Refined Methods of Analysis

Programs approved for use on LFD design projects are listed in Table 1.1-1 (LRFD listed in Table 1.1-2). The approval of these programs is subject to the following conditions and limitations:

1. While certain software packages provide design optimization and/or code compliance checks, these aspects were not included in the review and approval process. Acceptance has been based solely upon the review of generalized design forces (moments, shears, reactions, etc.) as calculated by the software.
2. Acceptance of a software package by the Department does not affect the responsibility of the user for the proper application of the software and interpretation of its results. The acceptance of a software package does not constitute an endorsement nor does it relieve the vendor and the designer from their responsibility for accurate, technically correct, and sound engineering results and services to the Department.
3. The Department's acceptance does not constitute any form of implied warranty, including warranty of merchantability and fitness for a particular purpose. The Commonwealth makes no warranty or representation, either expressed or implied, with respect to this software or accompanying documentation, including their quality performance, merchantability, or fitness for a particular purpose. In addition, the Commonwealth will not be liable for any direct, indirect, special, incidental, or consequential damages arising out of the use, inability to use, or any defect in the software or any accompanying documentation.

Only the version of a program listed in the tables below has been tested and approved. If any changes and/or modifications have been made to a program since its approval date, then re-approval of the program is required.

Table 1.1-1 - The List of Approved Computer Programs for the "3D or Refined" Analysis of Girder Bridges (for LFD design)

CONSULTANT	PROGRAM NAME	DATE SUBMITTED	DATE APPROVED
BSDI Coopersburg, PA	3D System	10/14/88	11/10/88
Modjeski and Masters, Inc. Harrisburg, PA	CURVBRG plus CGIS V4.2	10/17/88	11/10/88
SAI Consulting Engineers, Inc. Pittsburgh, PA	LOAD3D and SUPERSAP	12/12/88	12/27/88
Telos Technologies, Inc. Syracuse, NY	CBRIDGE	5/16/90	6/11/90

Table 1.1-2 - The List of Approved Computer Programs for the "3D or Refined" Analysis of Girder Bridges (for LRFD design)

CONSULTANT	PROGRAM NAME	DATE SUBMITTED	DATE APPROVED
SAI Consulting Engineers, Inc. Pittsburgh, PA	LOAD3DLRFD	2/27/08	11/12/08
BSDI Coopersburg, PA	3D v2.1 LRFD	6/25/10	8/4/11

2.0 COMPUTER PROGRAM ACCEPTANCE – GENERAL/SPECIFIC ANALYSIS

- a) STAAD/Pro 2006 has been tested and approved by the Department for general purpose structural analysis.

However, the following features of STAAD have not yet proven satisfactory to the Department and shall not be used until approved by the Chief Bridge Engineer:

1. The moving load generator.
2. The response spectrum and forced vibration analysis capabilities.
3. All design code check capabilities.
4. Analyses of live loads on 3-dimensional structural models.
5. Seismic design and analyses.

- b) VBent V2.3.0 has been tested and approved by the Department for Pier analysis (in addition to PAPIER).

- c) LUSAS v14 has been tested and approved by the Department for general purpose structural analysis.

However, the following features of LUSAS have not yet proven satisfactory to the Department and shall not be used until approved by the Chief Bridge Engineer:

1. The moving load generator.
2. The response spectrum and forced vibration analysis capabilities.
3. All design code check capabilities.
4. Analyses of live loads on 3-dimensional structural models.
5. Seismic design and analyses.
6. Analyses involving the non-linear behavior of material properties.

3.0 COMPUTER PROGRAM ACCEPTANCE - SEISMIC ANALYSIS

WinSEISAB v5.1.0, having been tested and approved by the Department, shall be used for seismic design and analysis of girder bridges, where applicable.

4.0 COMPUTER PROGRAM ACCEPTANCE - GEOTECHNICAL

The following programs have been tested and approved for use on department projects:

1. GRLWEAP 2003 - Pile hammer analysis (refer to section D10.7.3.8.4aP)
2. Driven V1.2 - Ultimate Vertical Static Pile Capacity (refer to FHWA Report SA-98-074, A10.7.3.8.6 and D10.7.3.8.6)
3. GSTABL7 with STEDwin - Slope stability analysis (refer to Pub 293 and D10.6.2.5)
4. COM624P and/or LPILE 5.0 (refer to D10.7.3.12.2P)
5. SLIDE V6.0 - Slope stability analysis (refer to Pub 293 and D10.6.2.5)

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APPENDIX K – APPROVED PREFABRICATED RETAINING WALLS

The following companies produce PennDOT approved prefabricated structural retaining walls:

The Reinforced Earth Company
8614 Westwood Center Drive
Suite 1100
Vienna, VA 22182

Prefabricated T-Wall Retaining Wall System
The Neel Company
8328-D Traford Lane
Springfield, VA 22152

Everwall Abutment and Retaining Wall Systems
Schuylkill Products Inc.
121 River Street
Cressona, PA 17929-1133

Dura-Hold Wall System
Dura-Sales
2481 Bull Creek Road
Tarentum, Pennsylvania 15084

Tricon Retained Soil Wall Panels
Tricon Precast, Ltd.
15055 Henry Road
Houston, Texas 77060

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**APPENDIX L – SOIL-CORRUGATED METAL STRUCTURE INTERACTION SYSTEMS
SOIL-CULVERT INTERACTION (SCI) DESIGN PROCEDURE**

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1.0 GENERAL

1.1P Scope

The Soil-Culvert Interaction (SCI) Design Procedure⁽¹⁾ includes provisions for the design of flexible corrugated metal structure installations with concrete relieving slabs and/or tension struts. The SCI procedure may only be used with the prior approval of the Chief Bridge Engineer.

1.2P Assumptions

The SCI design procedure is based on the following important assumptions which shall be addressed by the designer:

- (a) The structure shall be backfilled symmetrically.
- (b) No significant live loads shall be applied before the backfill reaches the crown of the structure.
- (c) No separate check of the buckling resistance is required if the structure is designed to meet all other requirements of the SCI procedure. Tests indicate that the buckling criterion will be satisfied provided the backfill is at least of the minimum quality provided by an ML or CL soil, in accordance with the USCS, compacted in lifts to 90% of the maximum dry unit weight as specified in ASTM D 1557 or AASHTO T 180.
- (d) The structural backfill shall extend for a minimum distance of 3 ft. or two-thirds of the span ($2/3 S$) on either side of the structure, whichever is greater. The structural backfill shall extend to the surface of the fill above the structure, or to a depth of cover of $2/3 S$, whichever is less.

The culvert section shall be designed for the maximum ring compression force, P_{max} , and the selected section shall be checked for structural adequacy against the following failure modes:

- (a) Seam strength
- (b) Handling stiffness (factory assembled pipe)
- (c) Moment capacity

1.3P Notation

The following notation shall apply for the design of buried metal structures by the SCI procedure:

AL	=	Axle load (kips)
B	=	Footing width (ft.)
C	=	Vertical distance from invert to crown (ft.)
C_1	=	Bearing constant dependent on soil type and strength (ksf) (Table 2.1.9-1)
C_2	=	Bearing constant (ksf/ft.) (Table 2.1.9P-1)
C_3	=	Bearing constant (ksf) (Table 2.1.9P-1)
C_4	=	Bearing constant for soil beneath footing (ksf) (Table 2.1.10P-1)
C_5	=	Bearing constant (kcf) (Table 2.1.10P-1)
C_6	=	Bearing constant (kcf) (Table 2.1.10P-1)
D_f	=	Footing depth (ft.)

D_s	=	Depth at inside of footing (ft.)
E	=	Young's modulus of the culvert (ksf)
E_s	=	Secant modulus of the soil backfill (ksf) (Table 2.1.4P-1)
F_p	=	Factor of safety
F_{pmin}	=	1.65 for the fill at the crown and no live load
F_s	=	Factor of safety against seam or wall compression failure
H	=	Depth of cover (ft.)
H_{RS}	=	Depth of cover measured to top of relieving slab (ft.)
I	=	Moment of inertia of the culvert ($\text{in.}^4/\text{in.}$) (See Appendix A12)
K_4	=	Line load factor (ft.) (Table 2.1.1P-1)
K_{T1}	=	Moment coefficient for culvert with struts
K_{m1}	=	Moment coefficient with zero cover (2.1.5P)
K_{m2}	=	Moment coefficient due to cover (2.1.5P)
K_{m3}	=	Moment coefficient due to live load (2.1.5P)
K_{m4}	=	Moment coefficient due to depth of soil cover and anticipated live load, or moment coefficient due to live load acting on concrete relieving slab (2.2.4P)
K_{mm}	=	Minimum moment coefficient based on experience with factory-fabricated pipe equal to 0.091 kip/ft.
K_{p1}, K_{p2}, K_{p3}	=	Ring compression force coefficients
$K_{p \min}$	=	Minimum acceptable plastic moment capacity for factory assembled pipe (kip-ft./ft.)
LL	=	Line load (AL/K_4) or line load equivalents for vehicles supported by concrete slab (kip/ft.) (Table 2.2.4P-1)
M_1	=	Maximum bending moment for $H = 0$ (kip-ft./ft.) (2.1.6P)
M_{1T}	=	Maximum moment for $H = 0$ (occurs at quarter-point and crown) with tension struts (kip-ft./ft.) (2.3.2P)
M_B	=	Maximum moment at quarter-point with final cover (kip-ft./ft.) (2.1.6P)
M_{B+L}	=	Maximum moment at quarter-point with live load over quarter-point (kip-ft./ft.) (2.1.6P) (Positive sign indicates compression for extreme fiber.)
M_{BT}	=	Maximum quarter-point moment at final cover with tension struts (kip-ft./ft.) (2.3.5P)
M_S	=	Quarter-point bending moment with slab in place (kip-ft./ft.)
M_{S+L}	=	Quarter-point moment with live load on slab (kip-ft./ft.)
M_p	=	Plastic moment capacity (kip-ft./ft.) (Table 2.1.3P-1)

$M_{p\min}$	=	Minimum acceptable plastic moment for factory fabricated pipe (kip-ft./ft.)
N_f	=	Flexibility number for zero cover
P_1	=	Quarter-point ring compression force for $H = 0$ (kip/ft.)
P_{1T}	=	Quarter-point ring compression force for $H = 0$ with tension struts (kip/ft.)
P_2	=	Quarter-point ring compression force with cover depth equal to H (kip/ft.)
P_H	=	Horizontal load on footing (kip/ft.)
P_{S+L}	=	Quarter-point ring compression force with live load on slab (kip/ft.)
P_V	=	Vertical load on footing (kip/ft.)
P_{all}	=	Allowable soil bearing pressure beneath footing (ksf)
P_{max}	=	Maximum ring compression force (kip/ft.)
P_p	=	Plastic load capacity (kip/ft.)
P_{ult}	=	Seam strength or wall strength (kip/ft.)
p	=	Vertical bearing pressure (ksf)
q	=	Horizontal bearing pressure (ksf)
R	=	Top rise or vertical distance from haunch level to crown of structure (ft.) (Figure 2.1.1P-1)
R_B	=	Reduction factor for moment due to backfill (2.1.5P)
R_L	=	Reduction factor for moment due to live load (2.1.5P)
R/S	=	Rise span ratio
r_a	=	Radius of adjacent section above or below corner (ft.) ($k = 0.8$ for r_a below corner; $k = 0.9$ for r_a above corner)
r_c	=	Corner radius (ft.)
S	=	Span (ft.) (Figure 2.1.1P-1)
S_u	=	Undrained shear strength (ksf)
T	=	Maximum tension strut force (kip/ft.)
t	=	Slab thickness (ft.)
α	=	Angle between culvert wall and vertical at footing (deg) (Figure 2.1.10P-1)
γ	=	Unit mass of backfill (kcf)
γ_s	=	Unit mass of relieving slab (kcf) (assumed to be 0.150 kcf)
Δq	=	Difference between soil bearing at the corner and adjacent to the corner (ksf)
Δq_{all}	=	Allowable difference in soil bearing pressure (ksf)

The dimensional units provided with each notation are presented for illustration only, to demonstrate a dimensionally correct combination of units for the design of flexible corrugated metal buried structures using the SCI procedure. If other units are used, the dimensional correctness of the equations should be confirmed. Base dimensions for the SCI procedure are presented in terms of kips and feet units. Service load and load factor procedures are presented in terms of pound and inch units.

2.0 DESIGN

2.1P Corrugated Metal Culverts

Corrugated metal culverts constructed without relieving slabs and tension struts shall be designed by the SCI procedure using the following steps.

2.1.1P Maximum Ring Compression Force

The maximum ring compression force shall be determined using the following relationship:

$$P_{\max} = K_{p1}\gamma S^2 + K_{p2}\gamma HS + K_{p3}LL$$

where:

$$K_{p1} = 0.2 (R/S)$$

$$K_{p2} = 0.9 - 0.5 (R/S)$$

$$\begin{aligned} K_{p3} &= 1.25 - (H/S) \text{ for } 0.25 < H/S < 0.75 \\ &= 1.00 \text{ for } H/S \leq 0.25 \\ &= 0.50 \text{ for } H/S \geq 0.75 \end{aligned}$$

For tandem axles spaced at less than one-third the span of the culvert, the value of AL used to calculate the equivalent line load shall equal the sum of the loads carried on both axles. Table 2.1.1P-1 shows value of K_4 . For definitions of R and S for various culvert types, see Figure 2.1.1P-1.

Table 2.1.1P-1 – Line Load Factors, K_4

Depth (ft.)	No Relieving Slab			Relieving Slab Over Crown (ft.)
	2 Wheels per axle (ft.)	4 Wheels per axle (ft.)	8 Wheels per axle (ft.)	
1	4.3	5.0	8.5	12.8
2	5.3	6.4	9.2	13.4
3	7.9	8.7	10.6	14.0
5	12.3	12.5	13.5	14.6
7	14.4	14.5	14.6	15.3
10	16.0	16.0	16.0	16.0
15	22.0	22.0	22.0	---
20	28.0	28.0	28.0	---
30	40.0	40.0	40.0	---

Source: Reference 1

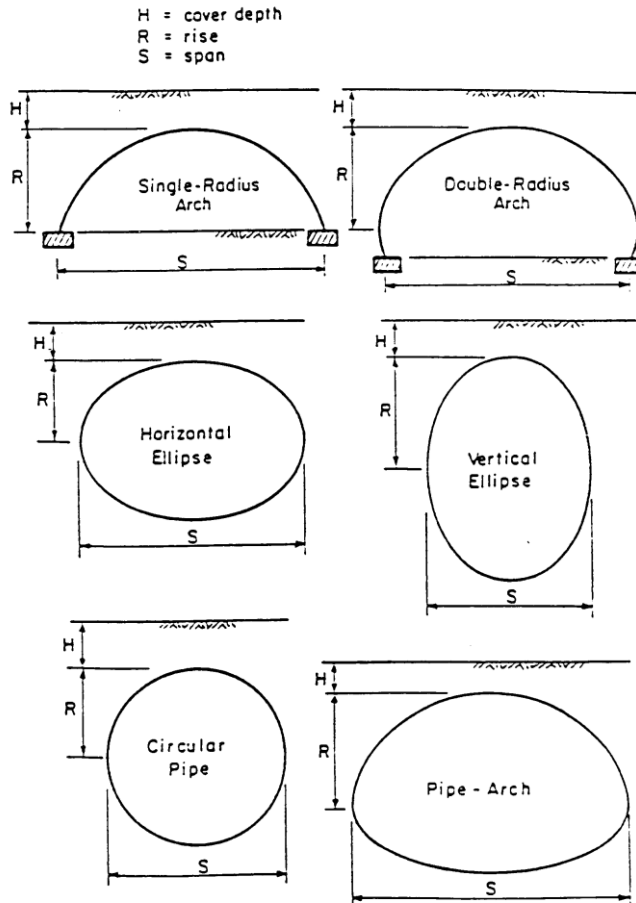


Figure 2.1.1P-1 - Typical Culvert Shapes

2.1.2P Seam Strength

The seam strength of the section shall be checked using the following relationship:

$$F_s = \frac{P_{ult}}{P_{max}} > 2.0$$

See AASHTO LRFD Bridge Design Specifications, 2010, Appendix A12, Tables A12-7, A12-8, and A12-15 for values of P_{ult} . For the computation of P_{max} , see 2.1.1P.

2.1.3P Handling Stiffness (Plastic Moment Capacity)

The culvert section shall be checked for handling and installation rigidity using the following relationship to calculate the minimum plastic moment capacity:

$$M_{p \min} = K_{mm} S$$

For values of M_p for corrugated aluminum and steel plate, see Table 2.1.3P-1 and the value of $K_{mm} = 0.091$ kip/ft based on experience for factory fabricated pipes.

Table 2.1.3P-1 - Properties of Corrugated Aluminum and Steel Plate

Plate Corrugation	Uncoated Thickness (in.)	Moment of Inertia $1 \times 10^{-4}(\text{ft.}^4/\text{ft.})$	Plastic Moment Capacity, M_p (kip-ft./ft.)	Plastic Load Capacity, P_p (kip/ft.)
Corrugated Aluminum: 2 2/3 in. x 1/2 in.	0.060	0.011	0.23	18.59
	0.075	0.014	0.29	23.22
	0.105	0.020	0.42	32.56
	0.135	0.026	0.54	41.85
	0.164	0.033	0.67	51.18
3 in. x 1 in.	0.060	0.050	0.54	21.36
	0.075	0.063	0.67	26.71
	0.105	0.089	0.95	37.43
	0.135	0.117	1.24	48.21
	0.164	0.145	1.52	58.99
6 in. x 1 in.	0.060	0.049	0.51	18.59
	0.075	0.062	0.64	23.22
	0.105	0.086	0.89	32.56
	0.135	0.112	1.15	41.85
	0.164	0.138	1.41	51.18
Corrugated Steel: 2 2/3 in. x 1/2 in.	0.060	0.011	0.39	31.0
	0.075	0.014	0.49	38.7
	0.105	0.020	0.69	54.3
	0.135	0.026	0.90	69.8
	0.164	0.033	1.11	85.3
3 in. x 1 in.	0.060	0.050	0.90	35.6
	0.075	0.063	1.12	44.5
	0.105	0.089	1.59	62.4
	0.135	0.117	2.06	80.4
	0.164	0.145	2.54	98.3

Source: Reference 1

2.1.4P Flexibility Number

The flexibility number accounts for the effects of soil-structure interaction. It shall be determined using the following relationship:

$$N_f = \frac{E_s S^3}{EI}$$

See Table 2.1.4P-1 for values of E_s . Use values of E equal to 1,468,800 ksf for aluminum and 4,176,000 ksf for steel. For section properties of steel and aluminum pipe, see AASHTO LRFD Bridge Design Specifications, 2010, Appendix A12, Tables A12-1, A12-2, A12-3, A12-4, A12-5 and A12-6.

Using the equation above, calculate N_f for each cover depth and live load condition which may result in peak stress conditions. It should be noted that N_f varies with the value of E_s . Values of E_s are presented in Table 2.1.4P-1. Load conditions, both during and after construction, shall be considered for each combination of cover depth and live load that may control the design.

Table 2.1.4P-1 - Secant Modulus of Backfill, E_s (ksf)

Backfill Material*	R/C ** (%)	Depth of Cover Over Quarter-Point, $(H+R)/2$ (ft.)***											
		2	4	6	8	10	12	14	16	18	20	22	24
ML & CL	90	31	33	33	33	33	33	33	33	33	33	33	33
	95	45	52	53	53	53	53	53	53	53	53	53	53
	100	60	71	74	75	75	75	75	75	75	75	75	75
SM-SC	90	35	45	50	53	54	55	56	56	57	57	57	57
	95	50	66	74	80	83	87	88	88	88	89	89	90
	100	104	141	162	175	184	190	194	197	199	201	202	203
GW, GP & SW, SP	90	25	29	32	33	35	36	37	37	38	39	39	39
	95	67	79	86	91	94	98	100	102	104	105	107	108
	100	141	168	184	196	205	212	218	223	228	232	235	238

*Unified Soil Classification System per ASTM

**R/C = Percent of the maximum dry unit weight in accordance with ASTM D 1557 or AASHTO T 180

***R = Rise of corrugated buried structure (ft.)

Value of H_{RS} is substituted for value of H when concrete relieving slab is used.

Source: Reference 1

2.1.5P Moment Coefficients

Moment coefficients are calculated for each appropriate loading condition. For the calculated value of N_f for the case of zero cover over the culvert, the moment coefficients K_{m1} and R_B shall be determined using the following relationships:

$$K_{m1} = \begin{aligned} &0.0046 - 0.0010 \log_{10} N_f \text{ for } N_f < 5000 \\ &0.0009 \text{ for } N_f \geq 5000 \end{aligned}$$

$$R_B = \begin{aligned} &0.67 + 0.87 ((R/S) - 0.2) \text{ for } 0.2 \leq R/S < 0.35 \\ &0.80 + 1.33 ((R/S) - 0.35) \text{ for } 0.35 \leq R/S < 0.50 \\ &2 R/S \text{ for } 0.50 \leq R/S \leq 0.60 \end{aligned}$$

For each of the anticipated loading conditions other than $H = 0$, the moment coefficients shall be determined using the following relationships:

$$K_{m2} = \begin{aligned} &0.018 - 0.004 \log_{10} N_f \text{ for } N_f < 5000 \\ &0.0032 \text{ for } N_f \geq 5000 \end{aligned}$$

$$K_{m3} = \begin{aligned} &0.120 - 0.018 \log_{10} N_f \text{ for } N_f < 100,000 \\ &0.03 \text{ for } N_f \geq 100,000 \end{aligned}$$

$$R_L = \frac{0.265 - 0.053 \log_{10} N_f}{(H/S)^{0.75}} \leq 1.0$$

2.1.6P Maximum Moment

The maximum moment will occur at both the quarter- point and the crown of the structure for the condition of zero cover and no live load. The maximum moment shall be determined using the following relationship:

$$M_1 = K_{m1} R_B \gamma S^3$$

The maximum moment for conditions of fill over the crown with live load occurs at the quarter-point. For each loading condition, determine the quarter-point moment using the following relationship:

$$M_{B+L} = M_B + K_{m3} R_L SLL$$

where:

$$M_B = M_1 - K_{m2} R_B \gamma S^2 H$$

For normal traffic loads up to H20 (i.e., axle loads to 32 kips), moments induced by live loads need only be considered where the cover depth is less than 20% of the span (i.e., H/S less than 0.20). Moments induced by axle loads up to 32 kips can be ignored for greater cover depths. For traffic or construction axle loads exceeding 32 kips, moments induced by live loads shall be considered for all heights of cover.

In no case shall any buried structure be designed with a depth of cover over the crown of less than 1 ft..

2.1.7P Quarter-Point Ring Compression Force

The quarter-point ring compression force for the condition of zero cover shall be determined using the following relationship:

$$P_1 = 0.95 K_{p1} \gamma S^2$$

The ring compression force for each additional anticipated loading condition shall be determined using the following relationship:

$$P_2 = P_1 + 0.95 (K_{p2} \gamma HS + K_{p3} LL)$$

2.1.8P Factor of Safety Against Development of Plastic Hinge

The development of a plastic hinge corresponds to mobilization of the yield stress throughout the section. For each loading condition including H = 0, the factor of safety against development of a plastic hinge shall be computed using the following relationship:

$$F_p = \left(0.5 \frac{P_p}{P} \right) \left(\sqrt{\left(\frac{M}{M_p} \right)^2 \left(\frac{P_p}{P} \right)^2 + 4} - \left(\frac{M}{M_p} \right) \left(\frac{P_p}{P} \right) \right)$$

The minimum factor of safety, $F_{p \min}$, against development of a plastic hinge is 1.65.

As an alternative to solving the above equation, the value of F_p can be determined from Figure 2.1.8P-1 using $M = M_1$ and $P = P_1$. When H is greater than 0, F_p can be determined using the above equation or Figure 2.1.8P-1 with $M = M_{B+L}$ and $P = P_2$.

See Table 2.1.3P-1 for values of M_p and P_p for corrugated aluminum and steel pipe.

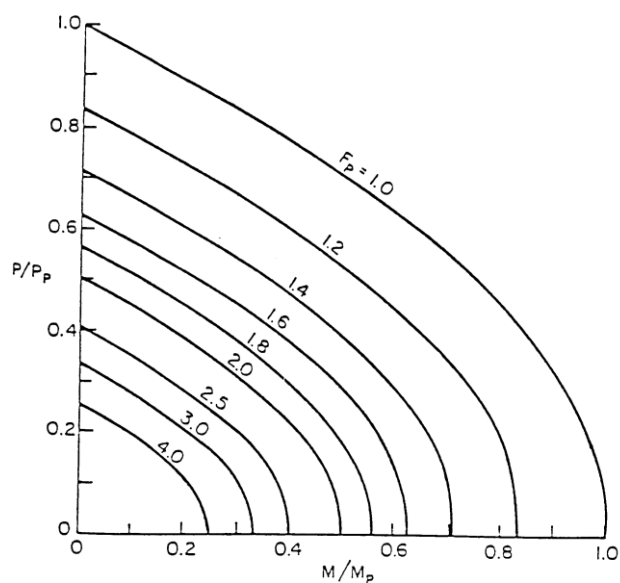


Figure 2.1.8P-1 - Factor of Safety Against Development of a Plastic Hinge

2.1.9P Difference in Soil Bearing Pressures (Pipe Arches Only)

$$\Delta q = P_{\max} \left[\left(\frac{1}{r_c} \right) - \left(\frac{1}{r_a} \right) \right] k$$

The value of Δq calculated using the above equation shall not exceed the value of q_{all} , as determined by the following relationships:

$$q_{\text{all}} < C_1 + C_2 (H + R)$$

$$q_{\text{all}} < C_3$$

See Tables 2.1.9P-1 and 2.1.9P-2 for values of the bearing constants C_1 , C_2 and C_3 .

Table 2.1.9P-1 - Lateral Bearing Constants, Pipe Arches and Arch Footings, Saturated Plastic Silts and Clays, Short-Term Undrained Conditions

Saturated Plastic Silts and Clays, Short-Term Undrained Conditions				
Soil Consistency	Undrained Shear Strength (ksf)	Bearing Constants		
		C_1 (ksf)	C_2 (ksf/ft.)	C_3 (ksf)
Soft	0.4	1.0	---	1.0
Medium	0.8	2.0	---	2.0
Stiff	1.6	4.0	---	4.0
Very Stiff	3.2	8.0	---	8.0

Source: Reference 1

Table 2.1.9P-2 - Lateral Bearing Constants, Pipe Arches and Arch Footings, Compact Soils

Compact Soils				
Soil Type*	Relative Compaction (%)	Bearing Constants		
		C ₁ (ksf)	C ₂ (ksf/ft)	C ₃ (ksf)
SW, SP, GW, GP	90	1.0	0.580	30
	95	1.5	0.770	40
SM, SC, GM, GC	90	2.2	0.456	25
	95	2.4	0.452	25
ML, CL, MH, CH	90	1.8	0.364	20
	95	2.0	0.400	25

*Unified Soil Classification System per ASTM

Source: Reference 1

2.1.10P Footing Bearing Pressure

Due to the behavior and nature of the connection between the corrugated metal plates and the footing, both lateral and horizontal stresses may be transferred to the soil backfill and subgrade (Figure 2.1.10P-1). The following relationships shall be used to determine the horizontal and vertical bearing pressures imposed by the culvert:

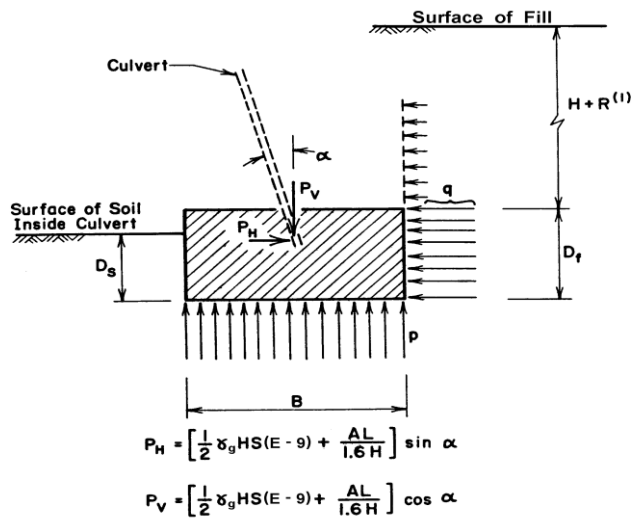
$$q = \left(\frac{P_H}{D_f} \right) - 0.5 \gamma (H + R)$$

$$p = \frac{P_v}{B}$$

where:

$$P_H = [0.5\gamma HS + (AL/1.6H)] \sin \alpha$$

$$P_v = [0.5\gamma HS + (AL/1.6H)] \cos \alpha$$



(1) $H + R$ = Height from foundations to springline for low- and high-profile arches

Figure 2.1.10P-1 - Loads and Pressures on Arch Footings

The value of q calculated above shall not exceed the allowable value for the type of soil adjacent to the footing as calculated according to 2.1.9P.

The value of p calculated above shall not exceed the allowable value calculated using the following relationship:

$$p_{all} = C_4 + C_5 B + C_6 D_s$$

See Tables 2.1.10P-1 and 2.1.10P-2 for bearing constants C_4 , C_5 , and C_6 . For definitions of P_H , P_V , $H + R$, D_f , B , D_s and α , see Figure 2.1.10P-1.

Table 2.1.10P-1 - Allowable Vertical Bearing Constants Arch Footings, Naturally Occurring Soil

Naturally Occurring Soil					
Soil Type*	Relative Density	Blow Count N (Blow/ft)	Bearing Constants		
			C_4 (ksf)	C_5 (ksf/ft)	C_6 (ksf/ft)
SW, SP, GW, CP	Loose	5	-	0.2	0.5
	Medium	12	-	0.5	0.8
	Dense	30	-	1.2	2.0
SM, SC, GM, GC	Loose	5	2.0	0.2	0.5
	Medium	12	3.0	0.5	0.8
	Dense	30	4.0	0.5	0.8
ML, CL, MH, CH Saturated	---	---	$P_{all} = 2S_u + \frac{D_s}{10}$		
ML, CL, MH, CH Unsaturated	---	5	1.0	0.2	0.5
	---	12	2.0	0.2	0.5
	---	30	3.0	0.2	0.5

Unified Soil Classification System

Source: Reference 1

Table 2.1.10P-2 - Allowable Vertical Bearing Constants Arch Footings, Compact Soils

Compact Soils				
Soil Type*	Relative Compaction (%)	Bearing Constants		
		C ₄ (ksf)	C ₅ (ksf/ft)	C ₆ (ksf/ft)
SW, SP, GW, GP	90	---	0.5	0.8
	95	---	1.2	2.0
SM, SC, GM, GC	90	3.0	0.5	0.8
	95	4.0	0.5	0.8
ML, CL, MH, CH	90	2.0	0.2	0.5
	95	3.0	0.2	0.5

*Unified Soil Classification System per ASTM

Source: Reference 1

2.2P Corrugated Metal Culverts with Concrete Relieving Slabs

Portland cement concrete relieving slabs (rigid pavements), as illustrated in Figure 2.2P-1, may be constructed over the top of corrugated metal culverts to reduce the moments induced in the structure by live loads. The design procedures described herein are a modified form of the SCI procedures presented in 2.1P to account for the effects of relieving slabs¹. The relieving slab shall extend a minimum of 5 ft. beyond the haunch of the culverts on each side, and shall cover all areas over the top of the culvert where traffic may travel, including trafficable shoulders.

The adequacy of the culvert design during construction (i.e., backfill up to the crown level and no live loading) shall be determined in accordance with the SCI procedures presented in 2.1P. The effects of live loads on the structure before the relieving slab is constructed shall be determined in accordance with the SCI procedure. The effects of live loads on the structure after the relieving slab is constructed shall be determined as presented below.

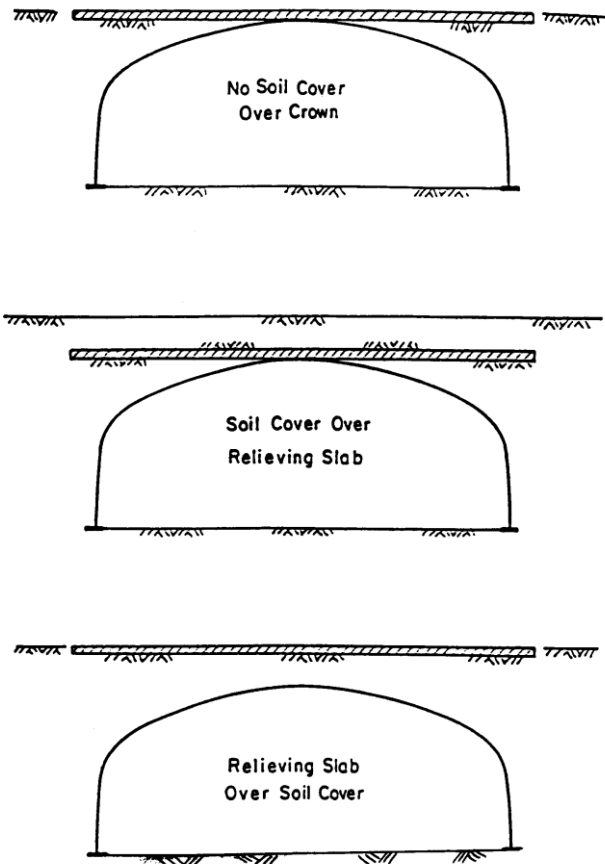


Figure 2.2P-1 - Metal Box Culverts with Cement Concrete Relieving Slabs

2.2.1P Flexibility Number

The flexibility number, N_f , shall be determined according to 2.1.4P. However, the depth of soil cover used to determine the secant modulus, E_s , of the soil backfill in Table 2.1.4P-1 shall be based on depth of soil cover, H_{RS} , measured from the top of the relieving slab.

2.2.2P Moment Coefficients

The moment coefficient for the condition of zero cover shall be determined using the following relationships:

$$K_{m2} = \begin{aligned} &0.018 - 0.004 \log_{10} N_f \text{ for } N_f < 5000 \\ &= 0.0032 \text{ for } N_f \geq 5000 \end{aligned}$$

The moment coefficient for live loading shall be determined using one of the following relationships:

$$K_{m4} = \begin{aligned} &0.024 - 0.005 \log_{10} N_f \text{ for } N_f < 25\,000 \\ &= 0.002 \text{ for } N_f \geq 25\,000 \end{aligned}$$

2.2.3P Bending Moment

The bending moment for the condition of zero soil cover shall be determined using the following relationship:

$$M_s = M_1 - R_B K_{m2} S^2 [\gamma_s t + \gamma (H - t)]$$

The moments and moment coefficients shall be determined in accordance with the equations in 2.1.5P and 2.1.6P.

2.2.4P Quarter-Point Moment

The quarter-point moment, M_{S+L} , for the dead and anticipated live load shall be computed using the following relationship:

$$M_{S+L} = M_s + K_{m4} SLL$$

The moment coefficient for live loading on a relieving slab shall be determined using one of the following relationships:

$$K_{m4} = 0.024 - 0.005 \log_{10} N_f \text{ for } N_f < 25\,000$$

$$= 0.002 \text{ for } N_f \geq 25\,000$$

Table 2.2.4P-1 shows values of LL.

Table 2.2.4P-1 - Line Load Equivalents for Vehicles Supported by Concrete Slabs

Axle Load of Design Vehicle (kips)*	Line Load, LL (kip/ft)**
10	0.8
20	1.6
30	2.3
32	2.5
40	3.1
45	3.5

*Total load on single axle or on both axles of a closely spaced tandem pair

**Produces same peak stress beneath slab as design vehicle

Note: For other axle loads, use linear interpolation.

Source: Reference 1

When the slab is constructed directly on top of the culvert, H is equal to the slab thickness, t . If there is a layer of fill above the crown and beneath the relieving slab, H will be greater than t .

2.2.5P Quarter-Point Ring Compression Force

The quarter-point ring compression force shall be determined using the following relationship:

$$P_{S+L} = P_1 + 0.95 \{ K_{p2} S [\gamma_s t + \gamma (H - t)] + LL \}$$

For the procedures to determine P_1 and K_{p2} , see 2.1.7P and 2.1.1P, respectively.

2.2.6P Factor of Safety Against Development of Plastic Hinge

The factor of safety, F_p , against development of a plastic hinge shall be determined in accordance with 2.1.8P or in accordance with Figure 2.1.8P-1 using $P = P_{S+L}$ (see 2.2.5P) and $M = M_{S+L}$ (see 2.2.4P). The calculated factor of safety against development of a plastic hinge shall satisfy the criterion presented in Figure 2.1.8P-1.

2.3P Corrugated Metal Culverts with Temporary Vertical Tension Struts

For cases in which bending moments from soil backfill will result in unacceptable deformation or stresses in the culvert during construction, tension strut may be used between the invert and crown to reduce bending moments and control deformations during backfilling, as illustrated in Figure 2.3P-1.

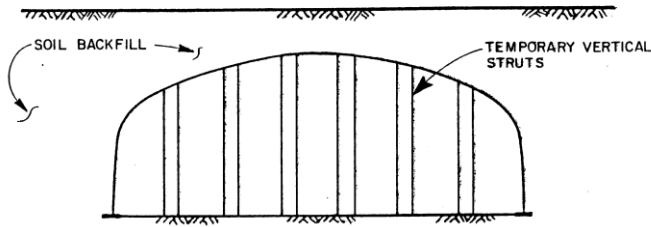


Figure 2.3P-1 - Corrugated Metal Culvert with Temporary Vertical Tension Struts

2.3.1P Maximum Tension Strut Force

The maximum tension strut force may be estimated using the following equation:

$$T = K_{T1} \gamma SC$$

where:

$$\begin{aligned} K_{T1} &= 0.125 - 0.025 \log_{10} N_f \text{ for } N_f < 10,000 \\ &= 0.025 \text{ for } N_f \geq 10,000 \end{aligned}$$

The value of N_f shall be determined in accordance with 2.1.4P.

Due to the possibility of the progressive failure of all tension rods if any individual rod should fail, it is recommended that the axial stress in the tension rods shall not exceed one-half the yield stress.

The spacing between tension rods shall not exceed 3 ft. to prevent excessive local deformation of the crown and invert.

2.3.2P Bending Moment

The maximum bending moment for soil backfill at the crown shall be determined using the following relationship:

$$M_{IT} = 0.67M_1$$

See 2.1.6P for the procedure to determine M_1 .

2.3.3P Quarter-Point Ring Compression Force

The quarter-point ring compression force shall be determined using the following relationship:

$$P_{IT} = P_1 + 0.5T$$

The values of P_1 and T shall be determined in accordance with 2.1.7P and 2.3.1P, respectively.

2.3.4P Factor of Safety Against Development of Plastic Hinge

The factor of safety, F_p , against development of a plastic hinge shall be determined in accordance with 2.1.8P using $P = P_{IT}$, $M = M_{IT}$ and $F_p \geq 1.65$.

2.3.5P Moment Due to Backfill

The maximum moment due to the final fill cover with tension struts in place shall be determined using the following relationship:

$$M_{BT} = R_T M_B$$

where:

$$\begin{aligned} R_T &= 0.67 + 2.2 H/S \text{ for } H/S < 0.15 \\ &= 1.00 \text{ for } H/S \geq 0.15 \end{aligned}$$

The value of M_B shall be determined in accordance with 2.1.6P.

3.0 REFERENCES

1. Duncan, J. M., and Drawsky, R. H., Design Procedures for Flexible Metal Culvert Structures, Report No. UCB/GT/83-02, Department of Civil Engineering, University of California at Berkeley, in cooperation with Kaiser Aluminum and Chemical Sales, Inc., 2nd ed., 1983

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION
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PART 4

VOLUME 2

**APPENDIX M – SOIL-CORRUGATED METAL BOX STRUCTURE INTERACTION SYSTEMS
SOIL-CULVERT INTERACTION (SCI) DESIGN PROCEDURE**

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1.0 GENERAL

Use of Soil-Culvert Interaction (SCI) method requires prior approval of the Chief Bridge Engineer.

1.1P Scope

Corrugated aluminum and steel box structures may be designed in accordance with the following procedures.

C1.1P

Corrugated metal box structures are used for applications where large flow areas are required with minimum available heights. The addition of circumferential, or lateral, reinforcing ribs, similar to the stiffening elements discussed in AASHTO LRFD Bridge Design Specifications, 2010, Article 12.7.4, allows the support of relatively large spans without the full benefit of the soil-structure interaction realized by the lateral restraint the backfill provides to the circular, elliptical and arch shapes.

1.2P Assumptions

See Appendix L, 1.2P.

1.3P Notation

In addition to the notation presented in Appendix L for the design of corrugated metal structures by the SCI method, the following notation shall apply for the design of corrugated metal box structures by the SCI procedure:

AL	=	Maximum single-axle load of design vehicle or, if larger, the maximum load carried by two load-sharing (tandem) axles (kips) (Table 2.1.1P-1)
F _B	=	Footing load due to dead load (kip/ft.)
F _L	=	Footing load due to live load (kip/ft.)
H	=	Height of cover over box as measured to the inside crown (ft.) (greater than H _{min})
H _{min}	=	Minimum cover depth equal to 1.4 ft. for all spans
K	=	Live load coefficient (1/ft.) (Table 2.1.4.1P-1)
K _{1B} , K _{2B} K _{3B}	=	Box culvert moment coefficients
K ₄	=	Line load factor (ft.) (Appendix L, Table 2.1.1P-1)
M _{CD}	=	Crown design moment (kip-ft./ft.)
M _{HD}	=	Haunch design moment (kip-ft./ft.)
M _{TB}	=	Sum of crown and haunch moments due to backfill (kip-ft./ft.) (2.1.2P)

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P_B	=	Crown moment coefficient
R	=	Live load footing reaction reduction factor (Table 2.1.4.1P-1)
R_{DL}	=	Dead load footing reaction (kip/ft.) (2.1.4.1P)
R_{HB}	=	Haunch moment coefficient (Figure 2.1.2P-1)
R_{LL}	=	Live load footing reaction (kip/ft.) (2.1.4.1P)
S	=	Span (ft.)
γ	=	Unit mass of soil backfill (kcf)
ΔM_{TL}	=	Change of crown and haunch moments due to live load (kip-ft./ft.) (2.1.2P)

The dimensional units provided with each notation are presented for illustration only, to demonstrate a dimensionally correct combination of units for the design of corrugated metal box structures. If other units are used, the dimensional correctness of the equations shall be confirmed.

2.0 DESIGN**2.1P Load Factors**

The following load factors shall be applied to the dead and live loads:

- (a) Dead loads: 1.5
- (b) Live loads: 2.0

C2.1P

The load and impact factors included in the design moments are appropriate for these flexible buried structures, which differ from conventional bridge structures in the effects produced by soil-structure interaction. The key values used in designs by this specification are the crown and haunch moments.

The specification utilizes calculated moments from computer analysis of conditions known to produce more conservative moments than the actual live loads and soils. Allowance for impact is incorporated in the values of line load used to develop the live load moments the specification requires. Calculated moments based on these equivalent line loads are longer than actual measured moments caused by moving vehicle loads. This is true, even in cases which the vehicles were made to jump as they passed over the structures by placing 5/8 in. thick wooden planks directly above the structures crowns. The equivalent line loads used in developing the design live load moments in the specification, thus, contain an allowance for impact effects that have been shown to be conservative by comparison with actual field measurements.

2.1.1P Live Loads

Vehicle loads are represented by equivalent line loads, LL, which, at the level of the crown of the culvert, produce

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the same peak stress as the wheel or axle loading. The value of LL shall be determined using the following relationship:

$$LL = AL/K_4$$

For tandem axles spaced at less than one-third the span of the culvert, AL is the sum of the loads carried on both axles. Table 2.1.1P-1 shows values of the axial loads for various vehicles. For values of K_4 , see Appendix L, Table 2.1.1P-1.

Table 2.1.1P-1 - Values of AL for Various Vehicle Types and Loading Conditions

Vehicle Type	AL	
HS25	40 kips	
20 yd ³ scraper	Empty: 43 kips	Fully Loaded 60 kips

Note: Axle load, AL (kips), of loaded wheel tractor scraper may be estimated by multiplying the rated carrying capacity (yd³) by 3. Axle load when empty is approximately 70% of the fully-loaded axle load.

Source: Reference 1.

2.1.2P Bending Moments

The sum of the crown and haunch bending moments due to loading from the backfill shall be determined using the following relationship:

$$M_{TB} = [K_{1B} \gamma S^3 + K_{2B} \gamma (H - H_{\min}) S^2]$$

where:

$$K_{1B} = 0.0053 - 0.00024 (S - 12) \text{ for spans between 8 and 26 ft.}$$

$$K_{2B} = 0.053$$

The change of the crown and haunch bending moments due to live loads shall be determined using the following relationship:

$$\Delta M_{TL} = K_{3B} LL S$$

where:

for spans under 20 ft.

$$K_{3B} = \frac{0.08}{\left(\frac{H}{S}\right)^{0.2}}$$

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for spans between 20 and 26 ft.

$$K_{3B} = \frac{0.08 - 0.002(S - 20)}{\left(\frac{H}{S}\right)^{0.2}}$$

The equations above are valid only for spans between 8 and 26 ft.

The design crown moment shall be determined using the following relationship:

$$M_{CD} = P_B (1.5 M_{TB} + 2.0 \Delta M_{TL})$$

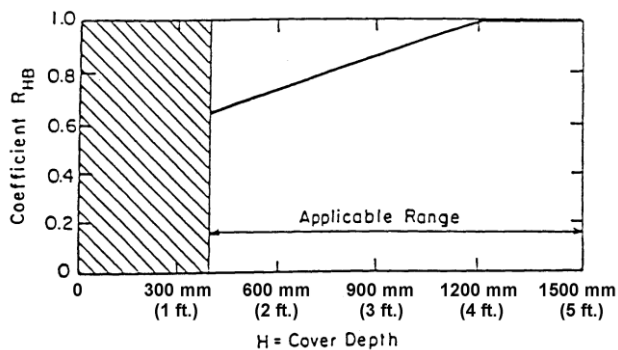
where:

$$\begin{aligned} P_B &= 0.55 \text{ to } 0.70 \text{ for spans under } 10 \text{ ft.} \\ &= 0.50 \text{ to } 0.70 \text{ for spans } \geq 10 \text{ ft.,} \\ &\quad < 15 \text{ ft.} \\ &= 0.45 \text{ to } 0.70 \text{ for spans } \geq 15 \text{ ft.,} \\ &\quad < 20 \text{ ft.} \\ &= 0.45 \text{ to } 0.60 \text{ for spans } \geq 15 \text{ ft.,} \\ &\quad < 26 \text{ ft.} \end{aligned}$$

The design haunch moment shall be determined using the following relationship:

$$M_{HD} = (1 - P_B) (1.5 M_{TB} + 2.0 R_{HB} \Delta M_{TL})$$

See Figure 2.1.2P-1 for values of R_{HB} as a function of the cover depth, H.



Source: Reference 2

Figure 2.1.2P-1 - Haunch Moment Coefficient

2.1.3P Footing Load

The footing load due to the weight of backfill shall be determined using the following relationship:

$$F_B = (0.025 \gamma S^2 + 0.5 \gamma S H)$$

The footing load due to the live load shall be determined using the following relationship:

$$F_L = R_{HB} LL$$

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The design footing load is the summation of the two equations above. Allowable footing pressures are given in Appendix L, 2.1.10P.

2.1.4P Footing Design

Footings for long-span corrugated metal structures shall be designed to support the loads imposed by the side walls of the structure and reduce the differential settlements which could cause load redistribution and excessive structural distortion. The footings for long-span structures shall be designed to preclude the possibility of a bearing failure into the foundation soils while allowing settlement relative to the adjacent backfill. The design of footings for long-span structures, other than corrugated metal box structures, shall be in accordance with AASHTO LRFD Bridge Design Specifications, 2010, Article 12.8.4. Metal box culverts shall be designed with concrete spread footings or a full steel invert.

2.1.4.1P Footing Reactions

The vertical component of the footing reaction for dead and live loads for long-span corrugated metal structures shall be determined using the following relationships:

- (a) Dead load:

$$R_{DL} = (0.025 \gamma S^2 + 0.5 \gamma S H)$$

- (b) Live load:

$$R_{LL} = AL K R$$

Table 2.1.4.1P-1 shows values of K and R. See Table 2.1.1P-1 for values of AL for various vehicle types and loading conditions.

Table 2.1.4.1P-1 - Live Load Coefficients for Corrugated Box Structure Design

Depth of Cover (ft)	Live Load Coefficients, K (1/ft)		Live Load Footing Reaction Reduction Factor, R
	Legal or Permit Vehicles*	Wheel Tractor-Scraper	
1	0.200	0.230	0.60
2	0.156	0.133	0.73
3	0.116	0.104	0.88
4	0.098	0.088	1.00
5	0.081	0.072	1.00
7	0.075	0.057	1.00
10	0.063	0.046	1.00

* Four tires per axle, single or tandem axles, 5 ft. maximum spacing of load-sharing tandem axle.

Source: Reference 1.

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2.2P Structural Plate Box Structures with Concrete Relieving Slabs

The procedure for the design of box structures with relieving slabs is similar to that used for structures without relieving slabs, except that the value of K_4 (line load factor) shall be determined using the values presented in Appendix L, Table 2.1.1P-1.

Where used, relieving slabs shall extend a minimum of 5 ft. beyond the haunch along each side of the structure and cover all areas over the top of the culvert where traffic may travel, including trafficable shoulders. Conditions with soil under or over the slab shall be considered the same for purposes of analysis.

3.0 REFERENCES

Duncan, J. M., and Drawsky, R. H., Design Procedures for Flexible Metal Culvert Structures, Report No. UCB/GT/83-02, Department of Civil Engineering, University of California at Berkeley in cooperation with Kaiser Aluminum and Chemical Sales, Inc., 2nd ed., 1983

Duncan, J. M., Seed, R. B., and Drawsky, R. H., "Design of Corrugated Metal Box Culverts", Transportation Research Record 1008, Transportation Research Board, Washington, DC, 1985

Kaiser Aluminum, Aluminum Box Culvert, DP-125, 8th ed., Kaiser Aluminum and Chemical Company, Oakland, CA, 1983

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PART 4

VOLUME 2

APPENDIX N – PROCEDURE FOR ISSUING A NEW STANDARD OR REVISING AN EXISTING STANDARD

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1.0 PROCEDURE FOR DEVELOPING A NEW STANDARD OR REVISING AN EXISTING STANDARD

DECISION FLOW CHART

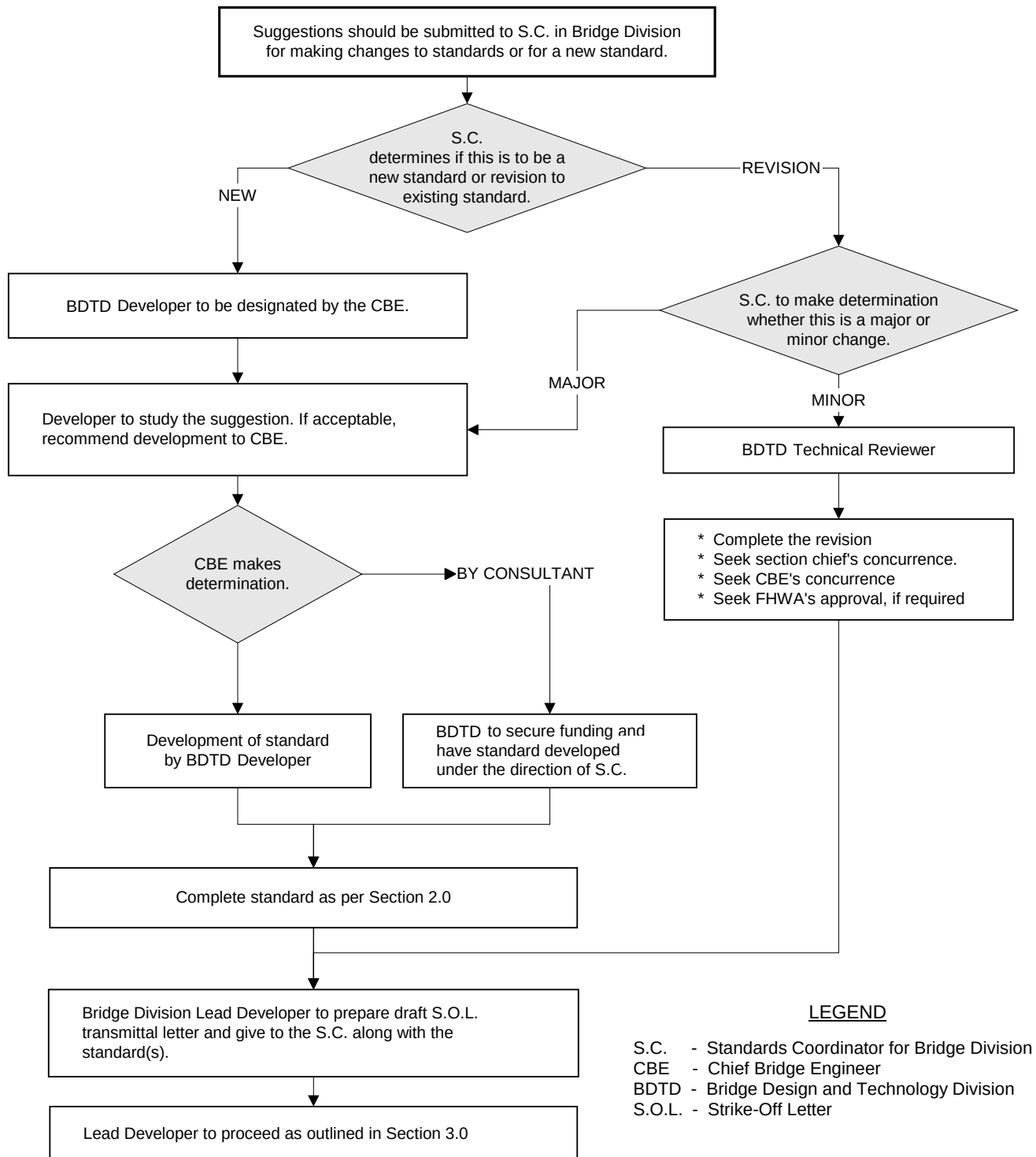


Figure 1.0-1 - Procedure for Developing a New Standard or Revising an Existing Standard

2.0 PROCEDURE FOR PREPARATION OF STANDARD DRAWINGS

1. Requests for a new Standard or revisions to an existing Standard are sent to the Bridge Design and Technology Division's Standard Coordinator (S.C.).
2. S.C. determines if submitted suggestion is to be a new Standard or a revision to an existing Standard.
3. S.C., along with input from the Chief Bridge Engineer (C.B.E.), considers the level of effort required to complete the requested Standard development/revision and, if necessary, contracts the assistance of a Design Consultant. Typically, Standards with minor changes are assigned to a designated BDTD Technical Reviewer or Standard Developer.
4. Reviewers and/or Developers create a red markup set of the Standard along with a description of change(s) and submit them to the S.C. for approval.
5. After approval by S.C., documents are given to Lead Developer/CAD Coordinator. Lead Developer determines whether to use Department or Design Consultant CAD resources based on the deadline and overall amount of current Department CAD work.
6. Lead Developer is responsible for sending out all new Standards and those Standards with significant changes via Clearance Transmittal Letter to Districts, Pennsylvania Turnpike Commission (PTC), FHWA and others for their review and comment. Acrobat "PDF" files of all documents are uploaded to the "Bridge Standards" folder on the Department's ftp site. An e-mail containing a link to the ftp folder is then sent to all reviewers on the Distribution list.
7. Clearance Transmittal Review comments, and both electronic and paper markups are logged in and compiled. These comments are reviewed by the appropriate Developers and final changes are prepared and submitted to S.C. for concurrence.
8. Lead Developer submits final markup set of Standard(s) to CAD unit for final changes.

3.0 PROCEDURE FOR ISSUANCE OF STANDARDS

Lead Developer to proceed as follows:

1. Obtain laser quality half-size drawings with date of drawing postdated by approximately one week.
2. Circulate Standard drawings, Transmittal Letter(s) and draft issuing Strike-off Letter (S.O.L.) to obtain signatures from the Chief Bridge Engineer and Director of the Bureau of Project Delivery.
3. Create Acrobat PDF files of signed drawing sheets and Transmittal Letter(s). Print out all drawing sheets and conduct final review. Upon satisfactory review, submit documents to both the Publications Office (Printing Lead Time of 4 weeks) and the Department's Internet Web Master.
4. Upon notification from the Web Master that the Web Site update is complete, a number, along with an effective date, will be assigned to the S.O.L. for release of the Standard. An Acrobat "PDF" file of S.O.L., including all of its attachments, is created.
5. S.O.L. is e-mailed to all individuals on Distribution list.
6. Publications Office sends PDF file of each Publication Change or New Edition to Graphic Services for printing and general distribution.

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PART 4

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APPENDIX O – SOIL-NAILED RETAINING WALLS

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SPECIFICATIONS

1.1 GENERAL

Soil-nailed walls may be used to stabilize and retain permanent or temporary cut slopes of weathered rock, granular soils, and clayey soils whose liquidity index is less than 0.2 and undrained shear strength greater than 1.04 ksf. The finished slope may be vertical or at a batter. At present, the permanent facing may be constructed as cast-in-place concrete, shotcrete, or with precast concrete panels.

Because of limited experience with these types of systems, they shall not be used under the following conditions:

- (a) For structures greater than 50 ft. in height
- (b) For retention of granular slopes composed of uniform fine sands or where 5 ft. cuts would not stay vertically open for the amount of time required for the installation of the nails and the application and subsequent curing of the shotcrete
- (c) For retention of cohesive clay slopes exhibiting liquidity indexes greater than 0.2 and undrained shear strengths less than 1.04 ksf
- (d) Where the nails would extend beyond the right-of-way limits
- (e) For retention, where high groundwater table would generate excessive flows
- (f) For retention of frost-susceptible and expansive soils
- (g) For retention of weathered rock with weak structural discontinuities that are inclined steeply toward and daylight into the excavation face

A service load design (SLD) approach as defined in the AASHTO Standard Specifications for Highway Bridges, fifteenth edition, (1992); AASHTO Interim Specifications (1993,1994); and DM-4 (1993), and an LRFD approach as defined in the 2010 AASHTO LRFD Bridge Design Specifications shall be used in the design of soil-nailed retaining walls. The most conservative design shall be used, i.e., the longest nail length, largest rebar size, etc.

Contractors specializing in the design and construction of soil-nailed structures shall be responsible for final wall design using the guidelines herein.

COMMENTARY

C1.1

Soil nailing systems are designed to reinforce in situ soil, using passive reinforcements to retain excavations or stabilize or construct vertical or nearly vertical slopes. In soil-nailed retaining structures, the inclusions (nails) are generally steel bars or other metal elements, which can resist tensile stresses and bending moments. They are either placed in drilled boreholes and grouted along their total length or driven in the ground. The nails are not prestressed, and their center-to-center spacing (density) is relatively tight, thus providing an anisotropic cohesion. The outside facing of the structure, which ensures local stability between the reinforcement layers, can consist of a thin layer of shotcrete 4 in. to 6 in. thick reinforced with a steel mesh, prefabricated panels, or a cast-in-place concrete veneer. Certain methods of nail installation may be proprietary, as well as certain types of prefabricated facings.

Within Appendix O, the 1992 AASHTO Standard Specifications for Highway Bridges and the 1993 and 1994 AASHTO Interim Specifications will be referred to collectively as AASHTO 92.

NCHRP Report 701, Proposed Specifications for LRFD Soil-Nailing Design and Construction was published and under review at the time of this writing.

The design of soil-nailed walls is a complex problem of soil-structure interaction, strongly influenced by methods of construction. To provide guidance in the design and construction of soil-nailed structures, the Federal Highway Administration (FHWA) has developed a design manual for the design and construction of soil-nailed walls (Byrne, et al, 1998). Preliminary design guidelines, restrictions, and technical considerations developed in earlier editions of this manual have been incorporated herein.

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1.2 LOADING

Soil-nailed retaining walls shall be investigated for all applicable load combinations from AASHTO 92, Section 3.22, and D3.4.1.1P-3 including, as a minimum:

- lateral earth and water pressures, including any live and dead load surcharge
- the self weight of the wall

Walls shall be designed for a minimum live load surcharge equal to 3 ft. of soil, or the actual surcharge, whichever is greater.

In general, temperature and shrinkage deformation effects are not applicable in the design of soil-nailed retaining walls. However, temperature and shrinkage deformation effects shall be considered in the application of soil-nailed retaining walls to underpass widening through end slope removal under an existing bridge abutment.

1.3 MOVEMENT UNDER THE SERVICE LIMIT STATE

The effects of horizontal and vertical wall movement on existing structures or underground utilities shall be investigated.

Horizontal and vertical displacements associated with the construction of the wall may be estimated using Figure 1.3-1. For calculations of settlement behind the wall, linear interpolation shall be used with a maximum value of δ_v at the top of the displaced wall decreasing to a value of zero at a distance of $\delta_h + \lambda$ from the back of the displaced wall as shown in Figure 1.3-1.

In addition to the displacements associated with the construction of the wall, any time dependent displacements of the wall shall be considered.

When precast panels, CIP concrete, or a second layer of shotcrete is added to the initial construction facing to provide a permanent facing, additional vertical settlement of the wall shall be considered.

C1.3

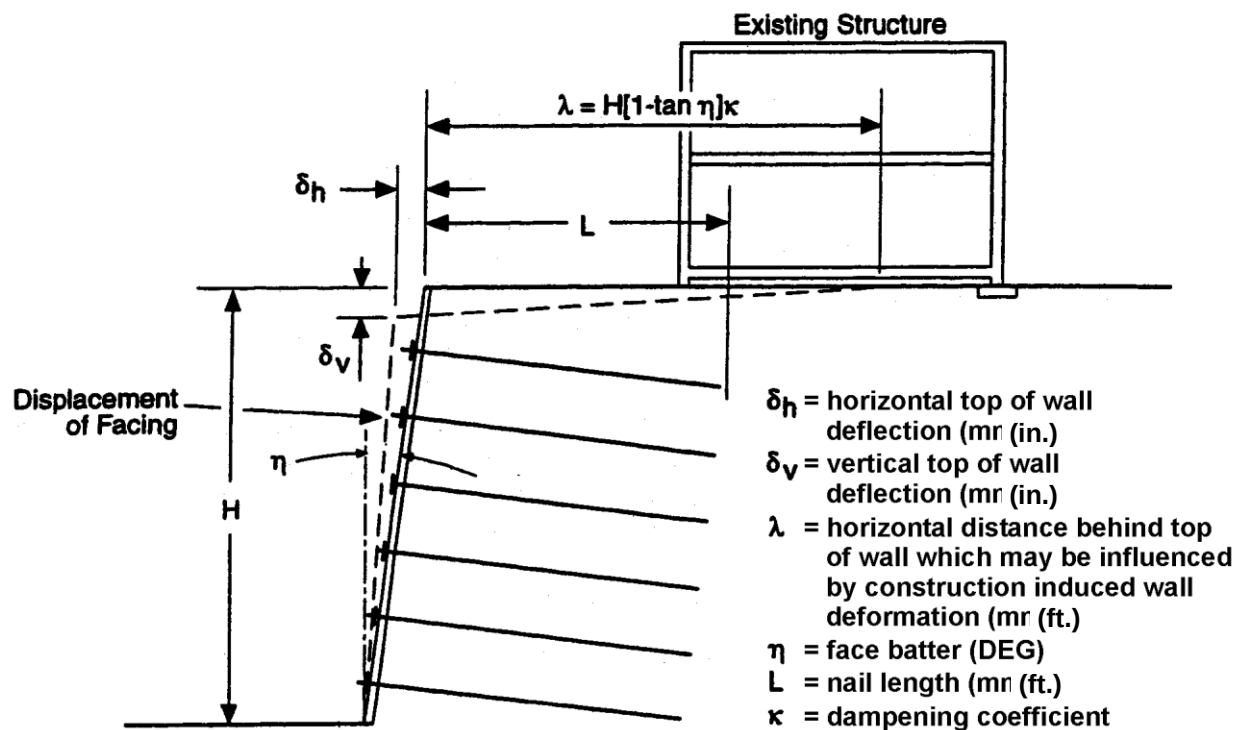
The horizontal and vertical movements of the wall given in Figure 1.3-1 are associated with the construction of the wall. These displacements are the result of the mobilization of the tensile loads within the nails.

Depending on the ground type, post-construction monitoring of wall displacements has indicated that some ongoing movements may occur with time.

For typical construction facings consisting of shotcrete 4 in. thick, the weight of the construction facing is supported by the installed nails. When thicker shotcrete facings are employed during construction, strut nails are usually installed to carry the additional weight. However, when permanent facing is installed after the initial construction facing has been applied, no provisions are made to carry the additional weight of the permanent facing. This additional weight is carried by the soil beneath the wall resulting in additional settlement and an increased potential for a bearing capacity failure of the soil.

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Type of Soil	Weathered Rocks Stiff Soils	Sandy Soils	Clayey Soils
$\delta_h - \delta_v$ coefficient κ	$H / 1000$ 0.8	$2 H / 1000$ 1.25	$3 H / 1000$ 1.5

Figure 1.3-1 - Deformation Behavior of Soil-Nailed Walls (after Clouterre, 1991)

1.4 SAFETY AGAINST SOIL FAILURE

The nailed soil mass shall be treated as a gravity wall and checked for stability against sliding, bearing capacity failure, overturning, and deep-seated foundation failure.

In determining safety against soil failure, the magnitude, location, and inclination of the resultant earth pressure load, applied to the back of the nailed soil mass, shall be taken as specified in Section 5.8.2 of AASHTO 92 and DM-4 (1993) and D11.6.3. However, the soil unit weight, γ , and friction angle, ϕ , shall be based on the in-situ soil behind the reinforced soil mass. The back and base of the nailed soil mass shall be defined as shown in Figure 1.4-1.

C1.4

The consideration of checking the dimensioned structure as a coherent structure subject to sliding and overturning forces is consistent with the present German Design Code, the only known adopted code of practice as outlined by Stocker, et al, (1979). The additional check on bearing capacity failure and deep-seated foundation failure is in accordance with the Manual for Design and Construction Monitoring of Soil Nail Walls (Byrne, et al, 1996).

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COMMENTARY

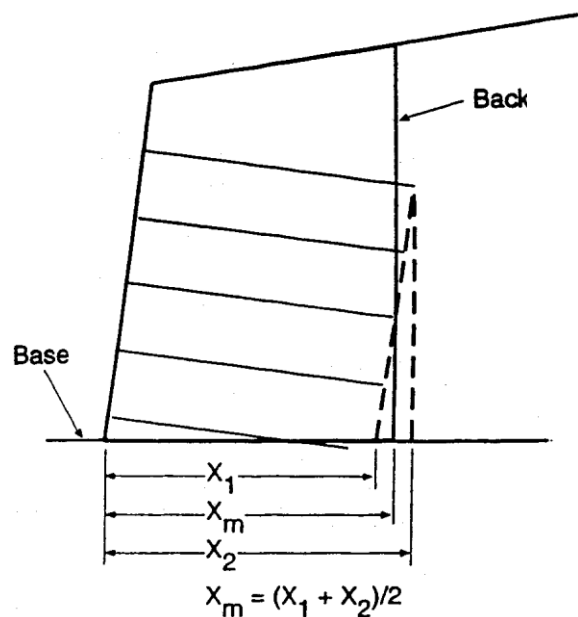


Figure 1.4-1 - Nailed Soil Mass (Byrne, et al, 1996)

For stability computations, live load surcharges shall be applied from a vertical plane beginning at the back of the nailed soil mass. In addition, a saturated soil condition shall be considered in determining stability.

With the following revisions, Figures 5.8.2A, 5.8.2B, and 5.8.2C in AASHTO 92 illustrate stability calculations applicable to soil-nailed walls:

- $L = X_m$
- Revisions as specified in DM-4 (1993), Section 5.8.2.

1.4.1 Sliding

The soil-nailed wall shall be dimensioned to ensure that the factor of safety against sliding specified in DM-4 (1993), Section 5.8.1 and D11.6.3.6 are satisfied. The vertical force, V_2 , due to the surcharge load shall not be used in determining the factor of safety against sliding.

1.4.2 Bearing

Bearing capacity shall be evaluated in accordance with Section 5.8.3 of AASHTO 92 and DM-4 (1993). Bearing resistance shall be evaluated in accordance with D11.6.3.2. However, the width of the footing for ultimate bearing capacity shall be taken as shown in Figure 1.4-1.

When precast panels, CIP concrete, or a second layer of shotcrete is added to the initial construction facing to provide a permanent facing, the soil beneath the wall facing shall be investigated for a local bearing capacity failure.

1.4.3 Overturning

For stability against overturning, the soil-nailed wall shall be dimensioned to ensure that the factor of safety specified in DM-4 (1993), Section 5.8.1, is satisfied, and the location of the

C1.4.2

Refer to the last paragraph of Section C1.3.

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resultant is in accordance with A11.6.3.3 and D11.6.3.3.

1.4.4 Overall Stability

Overall stability shall be evaluated in accordance with AASHTO 92, Section 5.2.2.3 and A11.6.2.3 and D11.6.2.3. For structures loaded with sloping surcharges, general stability analyses shall be performed using Swedish circle methods and yielding a minimum safety factor of 1.5.

1.4.5 Passive Resistance

The passive resistance of soil in front of the wall shall be neglected in stability calculations.

1.4.6 Nail Pullout Capacity**C1.4.6**

The ultimate pullout resistance per unit length used for preliminary design shall be taken as:

$$Q_u = 12\sigma_b\pi D \quad (1.4.6-1)$$

where:

Q_u = ultimate pullout resistance per unit length (kip/ft.)

σ_b = ultimate grout-ground bond stress estimated from Table 1.4.6-1, 1.4.6-2, or 1.4.6-3 (ksi)

D = nail drill hole diameter (in.)

The design pullout resistance in accordance with LRFD used for preliminary design shall be taken as:

$$Q = \phi_q Q_u r$$

where:

ϕ_q = ground pullout resistance factor
 = 0.7, Strength Limit State
 = 0.8, Extreme Limit State

Q = Design pullout resistance (kip/ft.)

r = soil weight factor, see Figure 1.5.1-3

In calculating the allowable pullout resistance, factors of safety shall be as specified in AASHTO 92, Section 5.7.6.2.

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Table 1.4.6-1 - Ultimate Bond Stress for Cohesionless Soils

Soil Type	Ultimate Bond Stress (ksi)
Non-plastic silt	0.0030 - 0.0045
Loess	0.0035 - 0.0110
Medium dense sand and silty sand/sandy silt	0.0070 - 0.0110
Dense silty sand and gravel	0.0115 - 0.0145
Very dense silty sand and gravel	0.0175 - 0.0345

Table 1.4.6-2 - Ultimate Bond Stress for Cohesive Soils

Soil Type	Ultimate Bond Stress (ksi)
Stiff Clay	0.0060 - 0.0085
Stiff Clayey Silt	0.0060 - 0.0145
Stiff Sandy Clay	0.0165 - 0.0290

Table 1.4.6-3 - Ultimate Bond Stress for Rock

Rock Type	Ultimate Bond Stress (ksi)
Marl/Limestone	0.0435 - 0.0580
Phillite	0.0145 - 0.0435
Chalk	0.0720 - 0.0865
Soft Dolomite	0.0580 - 0.0865
Fissured Dolomite	0.0865 - 0.1445
Weathered Sandstone	0.0290 - 0.0435
Weathered Shale	0.0145 - 0.0215
Weathered Schist	0.0145 - 0.0255
Basalt	0.0720 - 0.0865

Tables 1.4.6-1 through 1.4.6-3 are based on straight shaft nail drill holes formed by rotary drilling in rock and open hole construction in soils and subsequently grouting by gravity or low pressures.

Field pullout tests shall be conducted to verify the values of the ultimate bond stress used for preliminary design using

The tables of ultimate bond stress values are consistent with data available in the literature and shall not be used for final design without verification pullout tests as provided in the Special Provisions.

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procedures outlined in the specifications. Final design shall be based on field data obtained.

1.5 SAFETY AGAINST STRUCTURAL FAILURE

1.5.1 Global Stability

Soil-nailed structures shall be dimensioned to ensure a minimum factor of safety of 1.5 with respect to global stability.

The global stability of the soil-nailed wall shall be evaluated using slip surface limiting equilibrium methods of analysis modified to incorporate the additional resisting forces provided by the nail reinforcement. The reinforcing contribution of a nail shall be a function of the location at which the associated slip surface intersects the nail, as demonstrated in Figure 1.5.1-1 for a planar slip surface. The available nail strength is limited by the tensile strength of the nail, pullout resistance, or the strength of the nail head. Shear and bending of the nails shall be ignored. Figure 1.5.1-2 shall be used to determine the available nail strength as a function of the location of the intersection of the slip surface with the nail.

COMMENTARY

C1.5.1

The Davis (Shen, et al, 1981) or French (Schlosser, et al, 1984) method, modified to meet the requirements herein, may be used. In addition, in using the Davis method, the soil stress ratio, K , shall be taken as the at-rest earth pressure coefficient, K_0 . The French method is preferred where complicated soil stratigraphy is present or seepage pressures must be considered.

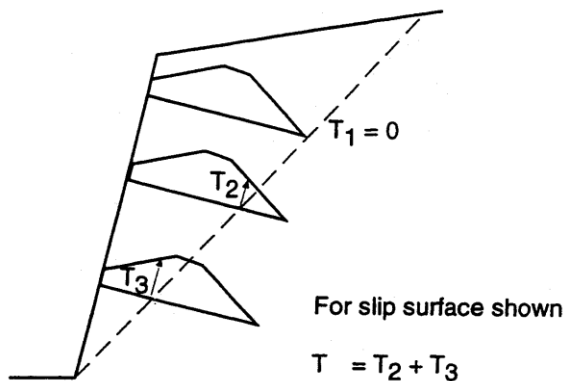
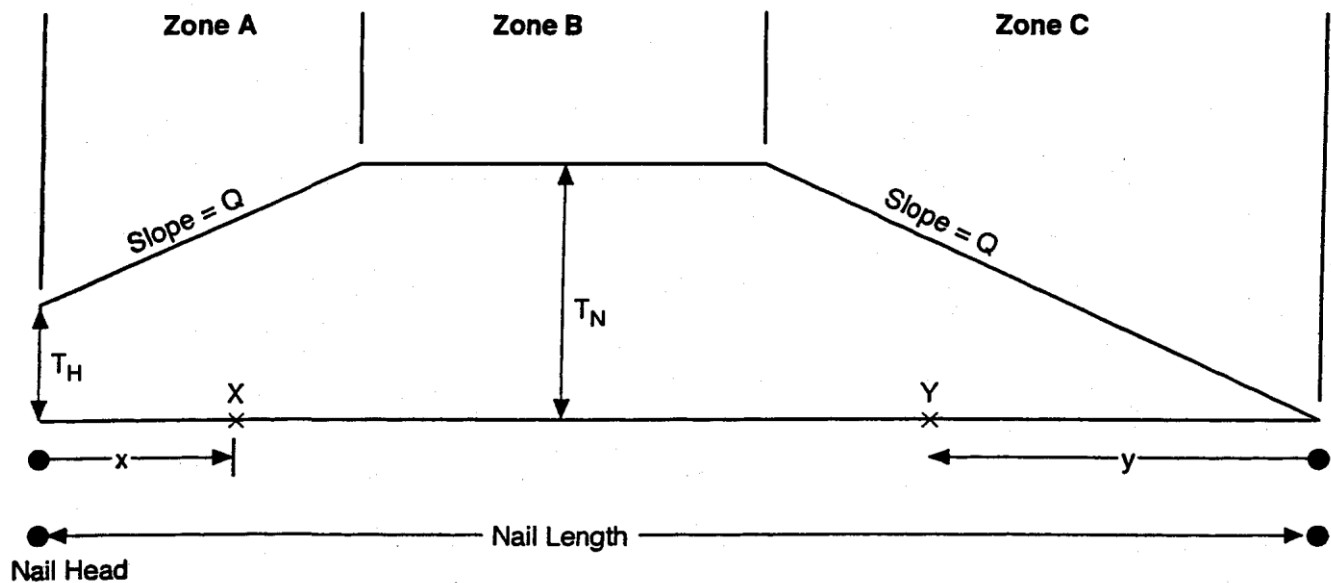


Figure 1.5.1-1 - Nail Reinforcing Contribution (Byrne, et al, 1996)

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Slip surfaces intersecting the nail in Zone A at Point X: $T = T_H + Qx$

Slip surfaces intersecting the nail in Zone B: $T = T_N$

Slip surfaces intersecting the nail in Zone C at Point Y: $T = Q_y$

T_H = allowable nail head strength in accordance with Section 1.5.2.3

T_N = allowable nail tensile strength as given by Equation 1.5.2.1-1

Q = allowable pullout resistance in accordance with Section 1.4.6

Figure 1.5.1-2 - Available Nail Resistance (after Byrne, et al, 1996)

For design calculation purposes, the nail length pattern used in the analysis of global stability shall be determined from Figure 1.5.1-3. However, it is preferred that nails of uniform length be installed for ease of construction. Shorter nails may be installed in the lower part of the wall provided the appropriate external stability checks are performed.

For LRFD design,

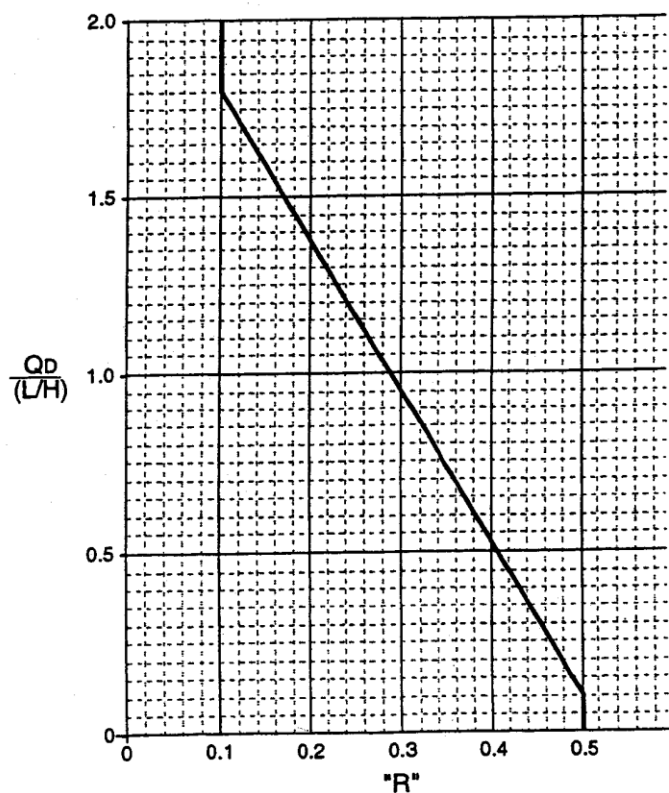
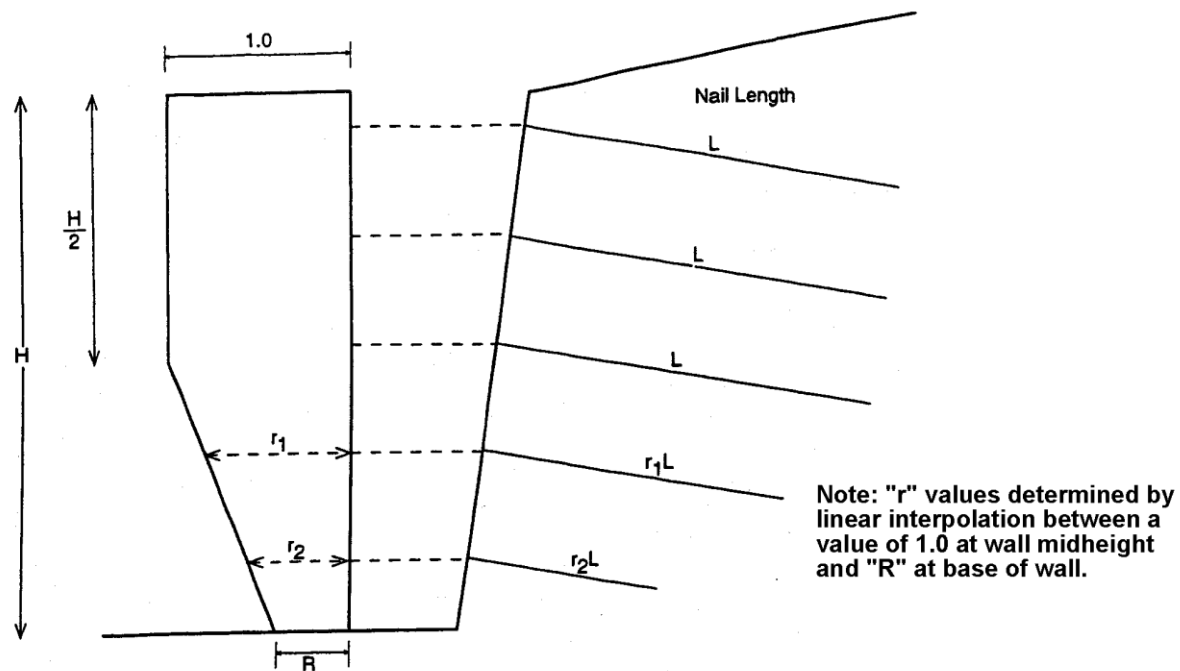
$$Q_D = (\phi_q Q_u) / (r_w \gamma S_u S_h)$$

where r_w = Soil weight factor from Table 4.6 of the Manual for Design and Construction Monitoring of Soil Nail Walls (Byrne, et al, 1996).

A limitation of the slip surface limit equilibrium method in the design of soil-nailed walls is that it is possible to define a wide variety of nail length patterns that satisfy the specified factor of safety, but result in excessive wall deflections. Performance monitoring of several soil-nail walls has demonstrated that the nails in the upper portion of the wall are more significant than those located lower down in the wall in developing resisting loads and controlling displacements. If nails having the same length as those located in the upper portion of the wall are used in the lower part of the wall, design calculations could overstate their contribution to the global stability of the wall. This can have the effect of indicating shorter nails and/or smaller nail sizes in the upper part of the wall, which is undesirable from a performance standpoint.

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L = maximum nail length (ft.)

H = wall height (ft.)

Q_D = dimensionless pullout resistance taken as:

$$Q_D = \frac{Q_u}{FS \gamma S_H S_V}$$

where:

FS = factor of safety as given in Section 1.4.6

Q_u = ultimate pullout resistance as given by Equation 1.4.6-1 (kip/ft.)

γ = unit weight (kcf)

S_H = horizontal nail spacing (ft.)

S_V = vertical nail spacing (ft.)

Figure 1.5.1-3 - Nail Length Distribution Assumed for Design (after Byrne, et al, 1996)

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When heterogeneous conditions exist, such as variable soil properties or highly non-uniform surface surcharges, critical slip surfaces that pass through points higher up on the wall shall be investigated.

Frictional and cohesive strength of soils to be used in stability analyses shall be obtained from direct shear or triaxial testing and evaluated at residual strength levels.

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For relatively uniform surface surcharges and homogeneous soil profiles, the critical slip surfaces resulting in the lowest calculated factors of safety will tend to pass through a point near the toe of the wall. However, when heterogeneities of any type are present, the critical slip surface may not pass through the toe of the wall. An example of this would be when a weak soil overlies a much stronger one. In this case, the critical slip surface may pass through the wall in the vicinity of the contact surface between the weak and strong soil as shown in Figure C1.5.1-1.

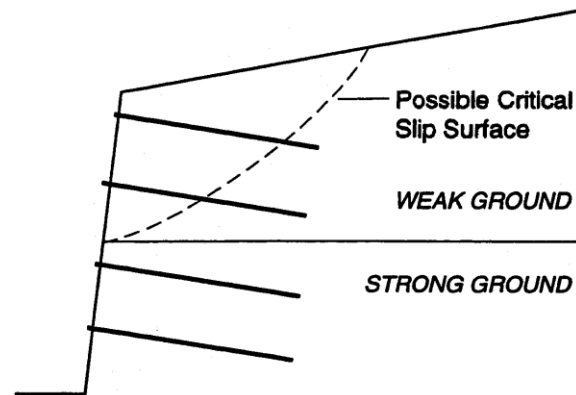


Figure C1.5.1-1 - Potential Critical Slip Surface for Heterogeneous Soil Profiles (Byrne, et al, 1996)

The stability of the wall during its construction shall be investigated. This investigation shall consider temporary construction conditions corresponding to the situation in which the next lift has been excavated, prior to the installation of the nails for that lift. A minimum factor of safety of 1.35 shall be provided.

1.5.2 Internal Stability

1.5.2.1 Soil Nail

The required horizontal component of each nail force shall be computed using the apparent earth pressure distribution given in Figure 1.5.2.1-1 and any other horizontal pressure components acting on the wall. The total nail force shall be determined based on the nail inclination. The horizontal/vertical nail spacing and nail capacity shall be selected to provide the required total nail force.

Construction conditions may control in situations where significant surcharge loads adjacent to the wall exist during construction.

C1.5.2.1

The apparent earth pressure distribution given in Figure 1.5.2.1-1 is based on field measurements of instrumented nails in several soil-nailed walls (Byrne, et al, 1996).

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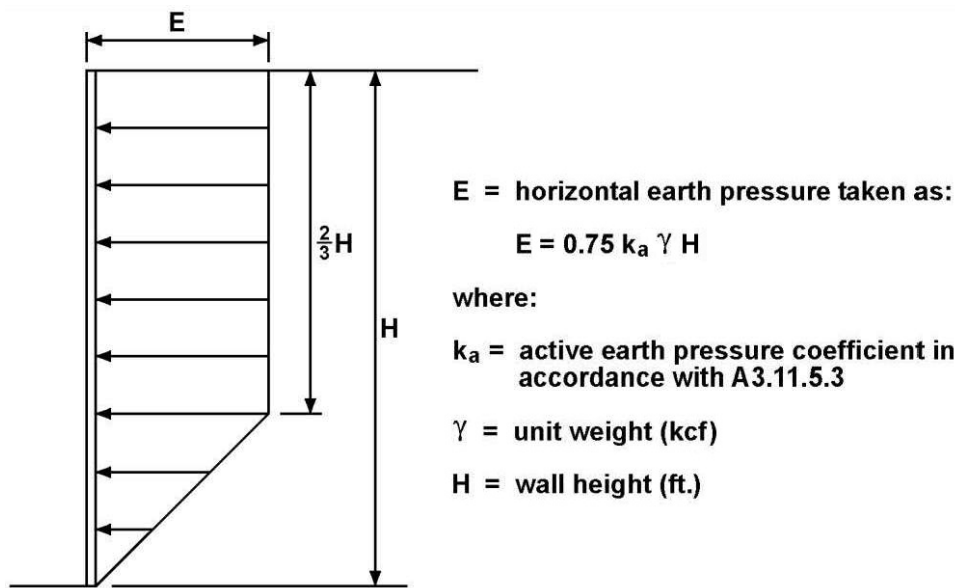


Figure 1.5.2.1-1 - Apparent Earth Pressure Distribution for Soil Nail

The allowable tensile capacity of the nail shall be taken as:

$$T_N = 0.55 A_N F_y \quad (1.5.2.1-1)$$

where:

T_N = allowable nail tensile strength (kips)

A_N = cross-sectional area of nail (in.²)

F_y = specified yield strength of nail (ksi)

For LRFD, nail tensile strength shall be taken as:

$$T_N = \phi_n T_{NN} \quad (1.5.2.1-1)$$

where:

T_N = nail tendon tensile strength (kips)

T_{NN} = Tendon yield strength (kips)
 $= A_N F_y$

ϕ_n = Tendon strength resistance factor
 $= 0.50$ Strength Limit State
 $= 0.67$ Extreme Limit State

Selection of nail inclination shall consider the location of suitable soil and rock strata and the presence of buried utilities and other geometric constraints.

Nail inclinations of 15 degrees are common. Flatter or steeper inclinations may be required due to drill rig access restrictions or to avoid underground obstructions. For tremie-grouted nails, care should be taken to ensure that grout fills the nail hole through the entire length of the nail for inclination angles of less than about 10 degrees due to the increased potential for voids in the grout column.

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Horizontal/vertical spacing of nails in soil shall not exceed 7 ft..

Minimum bar size used for soil nails shall be No. 6.

1.5.2.2 Soil-Nail Pullout

The nail load shall be developed by sufficient embedment beyond the line of maximum tension given in Figure 1.5.2.2-1.

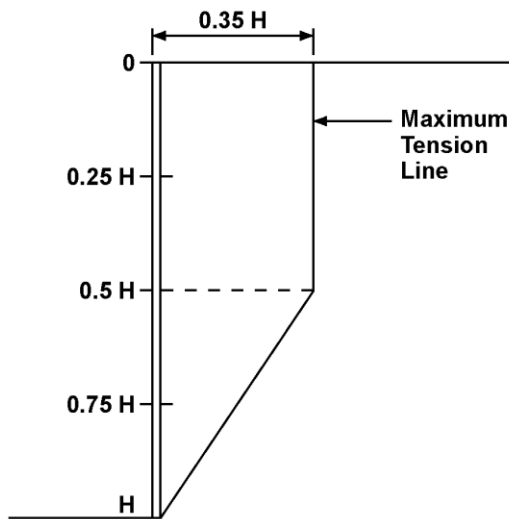


Figure 1.5.2.2-1 - Line of Maximum Tension

1.5.2.3 Nail Head

The allowable (or LRFD design) nail head strength shall exceed the computed nail head service load (or resistance) for all applicable loads including, but not limited to, horizontal earth pressure, surcharge, water pressure, and seismic loadings. Unless site specific monitoring information is available from walls constructed in similar soils, the nail head load due to horizontal earth pressure may be computed using the apparent earth pressure distribution given in Figure 1.5.2.1-1 multiplied by a nail head reduction factor, F_F , equal to 0.8.

The nominal nail head strength (or LRFD design nail head strength) will be limited by the flexural strength of the facing, the punching shear strength of the facing, or in the case of permanent wall facings, the tensile capacity of the headed studs.

The nominal nail head capacity based on the flexural strength of the facing shall be taken as follows:

$$(T_{HN})_F = C_F [(M_V)_{NEG} + (M_V)_{POS}] \left(\frac{8S_H}{S_V} \right) \quad (1.5.2.3-1)$$

where:

$(T_{HN})_F$ = nominal nail head capacity based on the flexural strength of the facing (kips)

COMMENTARY

Common nail spacings are 5 ft. by 5 ft. or 6 ft. by 6 ft..

C1.5.2.2

The shape of the maximum tension line is actually curvilinear and intercepts the surface at about 0.3H to 0.35H. The shape and location of the maximum tension line is empirically based and is applicable to nearly vertical walls with horizontal backslopes and homogeneous soil conditions. The line of maximum tension given in Figure 1.5.2.2-1 is an approximation to the curvilinear line and may not apply to heterogeneous soil conditions or sloped back surfaces.

C1.5.2.3

Researchers do not have a good understanding of the loads that develop at the nail head due to a lack of sound data in this area. However, the earth pressure distribution given in Figure 1.5.2.1-1 multiplied by a nail head reduction factor of 0.8 may be used until further research becomes available. The nail head reduction factor accounts for the fact that observed nail head loads for typical nail spacings have been significantly less than the loads developed within the nail at the maximum tension line.

The pressure factor, C_F , given in Table 1.5.2.3-1, accounts for the increased capacity of the facing due to soil arching and was taken as recommended by the FHWA (1996). Equation 1.5.2.3-1 also comes from the FHWA (1996).

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C_F = pressure factor for flexure given in Table 1.5.2.3-1

$(M_V)_{NEG}$ = vertical unit moment resistance at the nail head based on limit state design (kip-ft./ft.)

$(M_V)_{POS}$ = vertical unit moment resistance at midspan based on limit state design (kip-ft./ft.)

S_H = horizontal spacing of nails (ft.)

S_V = vertical spacing of nails (ft.)

When horizontal nail spacings are greater than the vertical spacing of nails or when horizontal unit moment capacities are less than those in the vertical direction, Equation 1.5.2.3-1 shall also be computed with unit moment capacities corresponding to the horizontal direction and with the vertical nail spacing substituted for the horizontal spacing and vice versa.

Table 1.5.2.3-1 - Pressure Factor for Flexure

Thickness of Facing	C_F for Temporary Facings	C_F for Permanent Facings
4 in	2.0	1.0
6 in	1.5	1.0
8 in	1.0	1.0

When using a C_F value greater than 1.0, the reinforcement ratio (based on gross area) shall not exceed 0.35 percent.

The nominal nail head capacity based on the punching shear strength of the facing shall be taken as:

$$(T_{HN})_v = 0.126 \sqrt{f'_c} \pi D'_c h_c \quad (1.5.2.3-2)$$

where:

$(T_{HN})_v$ = nominal nail head capacity based on the punching shear strength of the facing (kips)

f'_c = compressive structural design strength of concrete at 28 days, unless another age is specified (ksi)

D'_c = diameter of effective punching cone (in.)

- for bearing plate connections:

$$D'_c = b_{PL} + h_c$$

For reinforcing ratios (based on gross area) greater than 0.35 percent, Equation 1.5.2.3-1 tends to over estimate strength when C_F is greater than 1.0.

Equation 1.5.2.3-2 comes from the FHWA (1996). In determining the nail head capacity based on the punching shear strength of the facing, Equation 1.5.2.3-2 ignores the resistance provided by the soil as shown in Figure 1.5.2.3-1.

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where:

b_{PL} = width of bearing plate as shown in Figure 1.5.2.3-1 (in.)

h_c = effective cone depth equal to the full thickness of the facing as shown in Figure 1.5.2.3-1 (in.)

- for headed stud connections:

$$D'_c = S_{HS} + h_c$$

where:

S_{HS} = stud spacing as shown in Figure 1.5.2.3-1 (in.)

h_c = effective cone depth taken from the top of the headed studs as shown in Figure 1.5.2.3-1 (in.)

For headed-stud connections in which the length of the stud is less than half the stud spacing, the strength of the nail head based on the pullout capacity of individual studs shall also be evaluated.

In the case of permanent wall facings, the nominal nail head capacity based on the tensile strength of the headed studs shall be taken as follows:

Equation 1.5.2.3-3 was adopted from PCI (1999).

$$(T_{HN})_T = n A_{HS} F_y \quad (1.5.2.3-3)$$

where:

$(T_{HN})_T$ = nominal nail head capacity based on the tensile strength of the headed studs (kips)

n = number of studs

A_{HS} = cross-sectional area of stud body (in²)

F_y = yield strength of stud (ksi)

The nominal nail head capacity shall be the minimum determined from Equations 1.5.2.3-1, 1.5.2.3-2, and 1.5.2.3-3.

In determining the allowable nail head strength, the factor of safety shall be based on the limiting failure mode and taken as follows:

Factors of safety were obtained by dividing the critical load combination for horizontal earth pressure and live load surcharge by the appropriate resistance factor in accordance with LRFD specifications.

For nail head strength limited by:

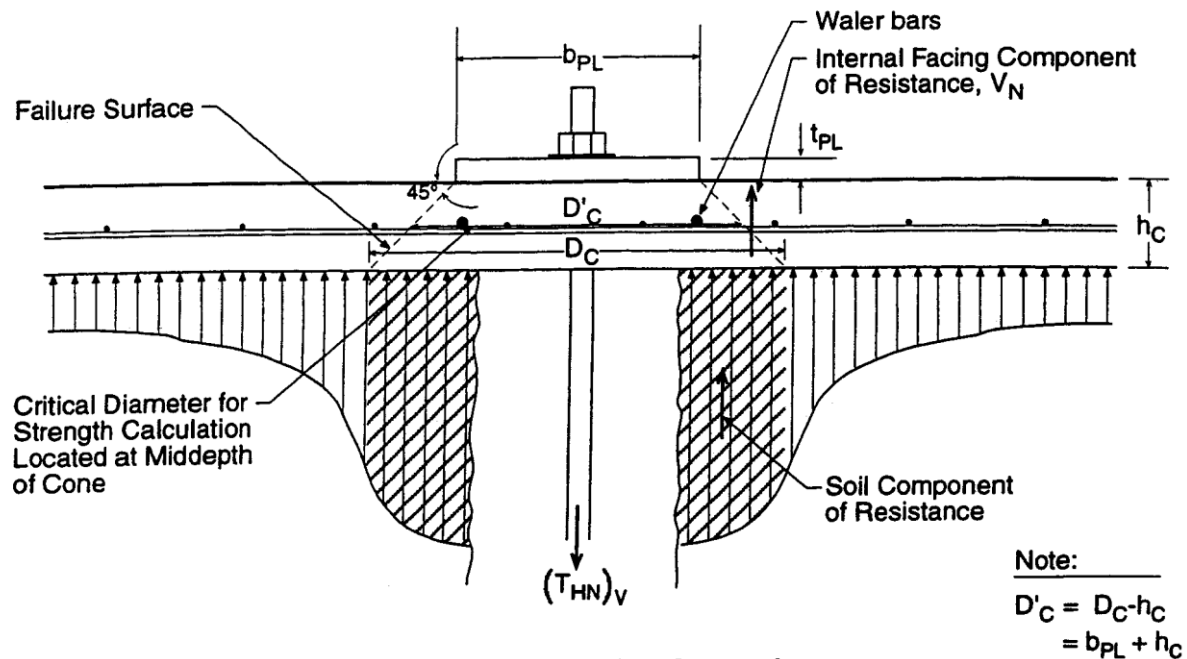
- facing flexure 1.75
- facing punching shear 1.75
- headed-stud yielding 1.75

For LRFD design, determine the design nail head strength

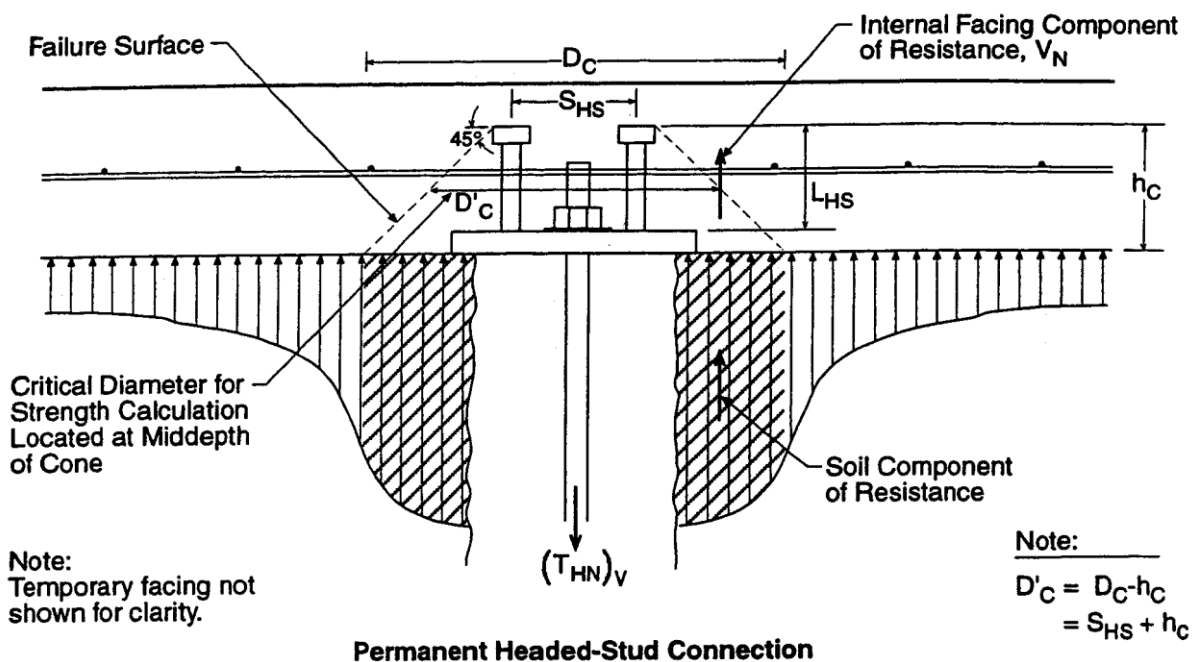
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by multiplying the nominal nail head strength by the corresponding resistance factors from Table 4.7 in the Manual for Design and Construction Monitoring of Soil Nail Walls (Byrne, et al, 1996).



Temporary Bearing-Plate Connection



Permanent Headed-Stud Connection

Figure 1.5.2.3-1 - Punching Shear of Nail-Head Connections (after Byrne, et al, 1996)

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1.5.3 Facing

A shotcrete facing reinforced with wire mesh may be used for all temporary support systems or for those permanent support systems where aesthetic and environmental concerns would permit it. For permanent applications, a cast-in-place curtain wall or precast panel shall be considered in front of the shotcrete facing.

Cast-in-place concrete facings shall be designed to carry the loads outlined in Section 1.5.2.3. The effects of the temporary facing shall be neglected. When precast concrete facings are used, the shotcrete facing shall be designed as the permanent structural facing. The precast panels and connections from the nail assemblies or shotcrete facing shall be designed to carry the load due to the weight of the facing and the pressure resulting from the previous drainage fill placed between them.

The minimum thickness of shotcrete facing for temporary support shall be 4 in.; for permanent support it shall be 6 in.. The minimum thickness of a cast-in-place facing shall be 8 in.. Minimum cover for mesh in the shotcrete facing shall be 2 in..

For temporary facings, a minimum of two No. 4 waler bars shall be placed along each nail row and shall be located between the face bearing plate and the back of the shotcrete facing.

For temporary facings, a minimum of two No. 4 waler bars shall be placed along each nail row and shall be located between the face bearing plate and the back of the shotcrete facing.

The moment and shear at the base (centerline of top nail row) of the upper cantilever portion of a soil-nail wall shall be investigated as illustrated in Figure 1.5.3-1. For fill placement and subsequent compaction behind the cantilever, an increased earth pressure shall be considered to account for compaction induced stresses behind the wall.

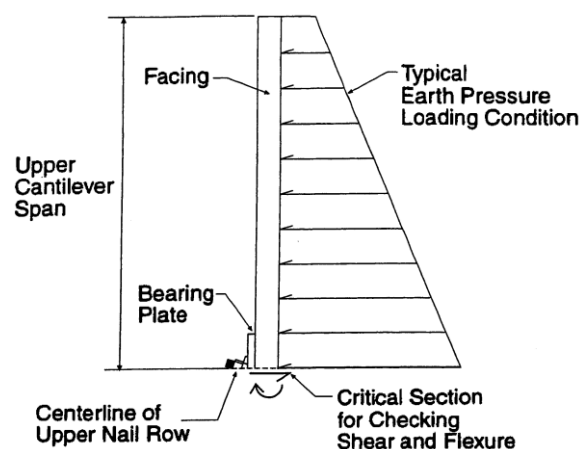


Figure 1.5.3-1 - Upper Cantilever Design Check (Byrne, et al, 1996)

The upper cantilever of a permanent soil-nail wall shall meet the requirements of Section 8.16.8.4 of AASHTO 92 and

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C1.5.3

Waler bars provide for the development of full plastic moment capacity and also provide an element of ductility in the event of a punching shear failure.

Unlike the rest of the wall, the upper cantilever cannot redistribute load by soil arching to adjacent spans, therefore, the moment and shear at the base of the cantilever needs to be checked.

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DM-4 (1993) for LFD designs, and D5.7.3.4 with A5.7.3.4 for LRFD designs.

The distance between the base of the wall and the bottom row of nails shall not exceed two-thirds the average vertical nail spacing.

Cantilevered end spans shall not exceed two-thirds the average nail spacing.

Shrinkage and temperature reinforcement requirements of Section 8.20 of AASHTO 92 and DM-4 (1993) and D5.10.8 with A5.10.8 shall apply for permanent facing systems.

For permanent applications using shotcrete, consideration shall be given to the introduction of expansion and contraction joints at intervals not exceeding those given in Section 5.5.6.5 of AASHTO 92 and DM-4 (1993) and D11.6.1.6.

1.5.3.1 Strut Nail

For construction facings thicker than 4 in., consideration shall be given to the installation of strut nails to support the weight of the facing.

1.5.3.2 Connection Systems

Bearing plates shall have a minimum width of 8 in. and a minimum thickness of 3/4 in..

Headed studs shall extend to at least the middepth of the permanent facing and their heads shall be anchored beyond at least one mat of reinforcement within the permanent facing.

For headed studs, the cross-sectional area of the head shall exceed 2.5 times the cross-sectional area of the body. In addition, the thickness of the head shall exceed one-half the difference between the head diameter and the body diameter.

1.6 SEISMIC DESIGN PROVISIONS

For typical soil-nailed walls constructed in Pennsylvania, forces arising from seismic activity need not be considered.

1.7 CORROSION PROTECTION

Nail head assemblies and nails shall be protected against

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If the requirement on the cantilevered end spans is not met, additional design checks are required in this area.

C1.5.3.1

For typical construction facings consisting of 4 in. of shotcrete, the soil nails are capable of supporting the weight of the facing through direct shear and bearing on the soil beneath the nails. The maximum thickness of shotcrete facing that can be supported by direct shear and bearing is dependent on the nature of the soil. For relatively thick facings constructed in less competent ground, the soil nails may not be able to support the weight of the facing without large downward and outward displacements of the wall.

C1.5.3.2

For bearing plates having a minimum yield stress of 36 ksi with dimensions less than the specified minimum, flexure and shear of the bearing plate may be critical.

C1.6

For peak ground accelerations below 0.25g, the seismic loading condition will generally not control in the design of soil-nailed walls (Byrne, et al, 1996). Since peak ground acceleration design values do not exceed 0.15g in Pennsylvania, seismic loading will seldom govern in the design of soil-nailed walls.

Should seismic forces be considered, the design engineer shall determine these forces with input from seismic experts with knowledge and experience in seismology and geotechnical engineering.

C1.7

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corrosion consistent with site conditions. The level and extent of corrosion protection shall be a function of the ground environment and of whether the nail is intended for temporary or permanent applications. For permanent applications in non-aggressive ground, a minimum of double corrosion protection shall be provided. Double corrosion protection can be achieved by resin bond epoxying of the nail and head assembly to a minimum thickness of 12 mils in accordance with AASHTO M 284/M 284M and providing minimum grout cover of 1.5 in. along the entire length of the nail. For permanent applications in aggressive ground or for critical structures, nails shall be fully encapsulated. Full encapsulation consists of a nail grouted full-length inside a plastic corrugated tube, placed in an oversized drill hole, and then grouted again against the side of the drill hole. Aggressive site conditions exist whenever one or more of the limiting values specified in Table D11.9.7-1 is exceeded. For all temporary applications, grout cover over the entire nail length shall be adequate. Other potentially corrosive conditions shall be identified and evaluated by the wall designer, and appropriate means of corrosion protection shall be designed by the soil nail wall specialty contractor.

The nail tendon protection, whether epoxy coating or encapsulation, shall extend at least 3 in. into the shotcrete construction facing.

The permanent facing shall provide minimum concrete cover over the nail head assembly in accordance with D5.12.3.

If the nail is encapsulated or is an epoxy-coated deformed bar with machine threads at the upper end, the corrosion protection is terminated to expose the bare tendon at the head of the nail in order to allow attachment of the bearing plate and nut. The portion of the exposed tendon within 3 in. of the retained earth is susceptible to corrosion; therefore, the corrosion protection shall extend for at least this distance into the shotcrete construction facing.

1.8 NAIL TESTING

Nail testing for ultimate capacity shall be conducted in accordance with Special Provisions of the Specifications for Soil-Nailed Retaining Walls.

1.9 DRAINAGE

Drainage systems shall be in accordance with A11.9.9 and D11.9.9.

In addition, measures shall be taken to control surface runoff and subsurface flow during construction.

1.10 SUBMITTALS

As a minimum for preliminary design, the information outlined for final design shall be provided with the following changes:

- (a) Special Provisions for Soil-Nailed Retaining Walls shall be amended as necessary and included.
- (b) Nails shall have no free stressing length.
- (c) Type of facing, or shotcrete or cast-in-place concrete

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(or both) shall be indicated.

As a minimum for final design, the following information shall be provided when applicable.

- (a) Subsurface exploration information . As a minimum, the following information shall be obtained through the subsurface exploration and testing program for use in design:
 - (1) Location and thickness of soil and rock units
 - (2) Engineering properties of in situ soil and rock and granular soil backfill including unit weight, shear strength, and compressibility
 - (3) Groundwater conditions
 - (4) Ground surface topography
 - (5) Geochemistry of soil and groundwater for corrosion potential
 - (6) Presence of stray electrical currents
- (b) Design earth pressures, water pressures, and surcharge loadings (to be included in final plan submission)
- (c) Allowable bearing pressures/bearing capacities (to be included in final plan submission)
- (d) Design depth of scour if the wall is located adjacent to a stream channel (to be included with foundation stage submission)
- (e) Geometric considerations including beginning and ending wall stations, wall profile and alignment, right-of-way limits, utility locations, construction considerations such as traffic restrictions or required construction sequences, and location of wall appurtenances such as drainage outlets, overhead signs and lights, and traffic barriers (to be included in the plans)
- (f) References and methods used for analysis including all calculations, computer analyses, assumptions, input, and explanation of all symbols, notations, and formulas (to be included in final plan submission)
- (g) Vertical wall element types, sizes, and spacing; hardware details; and erection sequence (to be included in the plans)
- (h) Details, dimensions, connections, and schedules of all reinforcing steel for vertical wall elements and facing

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(to be included in the plans)

- (i) Drainage requirements (to be included in the plans)
- (j) Corrosion protection and/or accommodation details for the wall elements and hardware (to be included in the plans)
- (k) Nail type and estimated capacity, nail inclination, and nail locations and spacings
- (l) Description of nail installation procedures including drilling and grouting
- (m) Corrosion protection details for the nails and nail head assemblies
- (n) Detailed plans for proof, performance/verification, creep (if applicable), and pullout testing of nails including specified load measuring devices, test locations, and testing procedures
- (o) Analyses of the stability of the wall at critical stages of construction

1.11 CONTRACTOR'S QUALIFICATIONS

See provisions and requirements under Construction Specifications.

1.12 CONSTRUCTION SPECIFICATIONS**1.12.1 General**

Construction specifications for contractor-designed soil-nailed walls shall be governed by the specifications herein. For soil-nailed walls designed by the Department, the following changes shall be made:

- (a) Section 1.12.2 – Delete “design and.”
- (b) Section 1.12.4(a) – Delete in its entirety and substitute, “Provide two sets of shop drawings, material certificates, construction procedures, and detailed construction sequencing plans. For temporary shotcrete facing, provide complete specifications for materials and methods.”
- (c) Section 1.12.4(g)(3) – Delete “Any modification of design or construction procedures shall be at no change in the contract prices” and substitute, “Should the nail tests prove that the ultimate bond stress actually being obtained in field production is significantly different than the ultimate bond stress assumed in design, the engineer will make design modifications to increase the nail length or decrease

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the nail spacing to ensure a stable completed structure. Additional nail lengths or nails beyond those shown on the plans will be paid under an item "Additional Nail Length."

1.12.2 Description

The furnishing of necessary design and materials as well as the construction of a retaining wall in a cut by internally reinforcing the soil mass with grouted reinforcing elements (nails).

The wall is constructed from the top down as the soil in front of the wall is removed and the nails are installed and grouted at each level. The exposed soil face is immediately protected with a wire mesh reinforced shotcrete facing. A structural cast-in-place concrete or precast concrete facing may subsequently be constructed and suitably attached.

1.12.3 Materials

- (a) Soil nails – Section 709.1 of Publication 408 – Thread as necessary. Provide epoxy-coated bars with a minimum thickness of 12 mils where required and shown on the plans. Epoxy coating to be in accordance with AASHTO M 284/M 284M.
- (b) Steel welded wire fabric – Section 709.3 of Publication 408.
- (c) Cast-in-place concrete – Provide Class AA concrete for structural concrete facing where required and shown on the plans, conforming to the requirements of Section 704 of Publication 408.
- (d) Precast concrete – Provide precast concrete facing where required and shown on the plans, conforming to the requirements for panels for mechanically stabilized earth walls.
- (e) Permanent structural shotcrete facing:
 - (1) Materials:
 - Cement – Cement must conform to the requirements of Section 701.1 of Publication 408 and ASTM C150 Portland Cement Type I and Type II.
 - Supplementary Cementing Materials – Silica Fume must conform to the requirements of Section 724.4 of Publication 408.
 - Water – All water used in shotcrete production is to be of drinking water standard and free of oil and chemical or

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organic impurities and meet the requirements of Section 720.1 of Publication 408.

Similarly, all water used for high-pressure water blasting, pre-dampening of substrate surfaces prior to application of shotcrete, in pressure sprayers for removal of rebound and overspray or green-cutting, and for shotcrete curing is to be of drinking water standards and free of oil and chemical or organic impurities.

- Aggregate – Use normal weight aggregates conforming to the requirements of Section 703.1 of Publication 408. Aggregates used are not to react with alkalis in the cement to an extent that results in excessive expansion of the shotcrete. Use normal 3/8" maximum size coarse aggregate combined with a cement concrete sand (Type A) to provide a blend, which conforms to the following composite gradation envelope.

Sieve Size	Total Passing Each Sieve, %
1/2"	100
3/8"	90 - 100
No. 4	70 - 85
No. 8	50 - 70
No. 16	35 - 55
No. 30	20 - 35
No. 50	8 - 20
No. 100	2 - 10

The 3/8" to No. 8 coarse aggregate fractions are to be stockpiled and added separately from the fine aggregate (nominal No. 4 maximum size) during batching operations.

- Admixtures – Do not use admixtures containing chlorides. Do not use shotcrete accelerators without written authorization by the Engineer.
 - a) Air-entraining admixtures must conform to the requirements of Section 711.3(d) of Publication 408.
 - b) Chemical admixtures, such as water reducers, high range water reducers (superplasticizers) and retarders must conform to the requirements of Section

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711.3(f) of Publication 408.

Additives are to be non-corrosive to the reinforcing steel and should prevent other detrimental effects such as cracking and excessive shrinkage.

- Reinforcement – Use Grade 60 reinforcing steel of the type, size and dimensions shown in the drawings, which conforms to the requirements of Section 709 of Publication 408.
 - Fibers – Add 0.06243 lb/ft³ of synthetic microfibers or glass fibers to the shotcrete for plastic shrinkage cracking control and enhanced resistance to sagging and sloughing. Fibers must conform to the requirements of Section 711.3(g) of Publication 408 and should be between 1/2 " and 1" in length.
- (2) Shotcrete quality – Produced by either dry mix or wet mix process achieving a minimum compressive strength of 4 ksi in seven days and 5 ksi in 28 days.
- (3) Pre-constructing testing – Implement a pre-construction trial to enable the Engineer to evaluate the ability of the proposed materials, shotcrete mixture, equipment and crew to produce shotcrete conforming to the project specifications. Acceptance of the pre-construction trial results by the Engineer is required prior to application of any shotcrete on the project.

The pre-construction trial is to be used to pre-qualify the nozzlemen proposed for use on the project. Nozzlemen who have not been pre-qualified are not permitted to apply shotcrete on the project. The pre-construction trial must use the same materials; shotcrete mixture and equipment proposed for use on the project and approximate actual working conditions, configuration, reinforcement and shooting positions as near as possible.

Nozzlemen must pre-qualify to shoot vertical and overhead work by separately shooting mock-ups. Five cores will be taken for core grading from locations directed by the Engineer, from the mock-up. Cores from each mock-up are to be core graded in accordance with the requirements of ACI C660, Shotcrete Nozzleman Certification. A test mock-up with

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any single core grade exceeding Core Grade 3 or with more than two of the five cores having a Core Grade 3 will be considered a failure. For a definition of shotcrete Core Grades see ACI C660.

Pre-qualify the shotcrete mixture by shooting a plain (non-reinforced) test panel with dimensions of 48" x 48" and 4" deep. The test panel is to be in accordance with ASTM C1140. Cure the test panel in the field, close to the location where shot, for seven days before being transported in the form to the testing laboratory. Cure the test panel under wet burlap covered with plastic sheet under field temperatures conditions. Protect the panels from disturbance or damage. Curing and transporting of test panels will be in accordance with ASTM C1140.

Assist the Department appointed testing laboratory by loading test panels, in their forms, onto their trucks. If the pre-construction test specimens fail to meet the project performance requirements, then make the necessary adjustments in shotcrete materials, mixture design or application, and re-shoot test panels. If the test specimens fail again after the adjustments were made, then switch to the form and pour method. No work is to commence on the project until the pre-construction performance testing requirements have been met.

- (4) Mixture proportions - Provide samples for testing. Submit for acceptance the recommended mixture proportion, source of materials, and all test results. Select mixture proportion on the basis of compressive strength tests of specimens continuously moist cured until tested at 28 days. For mixture acceptance purposes, average core compressive strength shall be equal to at least 1.2 times the required compressive strength specified in (2) above.
- (5) Batching, mixing and supply –
- A. Wet-Mix shotcrete shall be conform to one of the following methods:
- i. Central Mixing and Supply
- Aggregate, cement and silica fume are to be mass batched in a dry bagged form in accordance with the requirements of ASTM C1480. Water and chemical admixtures are

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to be capable of batching to the accuracy specified in ASTM C1141.

- Transit mixers are to be free of excessive accumulations of hardened shotcrete or concrete in the drum or on the blades. Blades are to be free of excessive wear. Transit mix delivery must conform to the requirements of ASTM C685.
- All shotcrete is to be shot within 90 minutes after addition of mix water to the batch. Shotcrete loads are to be of such batch size that this requirement is met. This time limit may be extended, subject to approval by the Engineer, if proper use is made of hydration controlling admixtures to maintain workability without re-tempering with water.

ii. Transit Mixing and Supply

- The same requirements for Central Mixing apply for except that all ingredients are to be added directly to the transit mixer, instead of the central mixer. Do not charge transit mixers to more than 70% of their rated capacity, to enable efficient mixing action.

iii. Volumetric Site Batching

- The mobile mixer unit for volumetric batching is to conform to the requirements of ASTM C685. The equipment is to proportion material to the tolerances specified in ASTM C685. The equipment is to be capable of thoroughly mixing dry bagged materials in sufficient quantity to maintain shotcreting continuity.
- Calibrate the equipment in accordance with the requirement of ASTM C685 at the start of every shift, or every 1000 cubic feet of shotcrete batched, whichever is more frequent, using a mass batch check of the volumetric proportioning.
- Feed systems for all materials dry bagged pre-mix (cement, silica

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fume, aggregates, admixtures) is to be interconnected so that if the feed stops, the system will stop.

- Clean the equipment thoroughly at least once per shift to prevent accumulation of aged material.
- Apply all wet-mix shotcrete within 90 minutes of mixing. Aged material is to be discarded and not applied in the work.

iv. Dry-Bagged Premix Supply

- The use of dry-bagged premix supply with water addition at the site will be permitted, provided the Contractor can demonstrate uniform mixing of the shotcrete and satisfactory conformance to all the specified performance requirements.
- Pre-dampening of dry-bagged material is a requirement.

B. Dry-Mix shotcrete shall be conform to one of the following methods:

i. Volumetric Site Batching

- The mobile mixer unit for volumetric batching must conform to the requirements of ASTM C685. The equipment must proportion materials to the tolerances specified in ASTM C685.
- The equipment must be capable of thoroughly mixing materials in sufficient quantity to maintain shotcreting continuity.
- The equipment must be calibrated in accordance with the requirement of ASTM C685 at the start of every shift, or every 1000 cubic feet of shotcrete batched, whichever is more frequent, using a mass batch check of the volumetric proportioning.
- Feed systems for all materials dry bagged pre-mix (cement, silica fume, aggregates, admixtures) must be interconnected so that if the feed stops, the system will stop.

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- Clean the equipment thoroughly at least once per shift to prevent accumulation of aged material.
- Apply all dry-mix shotcrete within 45 minutes of the first contact of cement with water. Aged materials are to be discarded and not applied in the work.

ii. Dry-Bagged Premix Supply

- The use of dry-bagged premix supply with water addition at the site will be permitted, provided part of mix water is added in pre-dampener, before discharge into the shotcrete gun.
- Contractor must demonstrate uniform mixing of the shotcrete and satisfactory conformance to all the specified performance requirements.

(6) Delivery equipment – the shotcrete delivery equipment shall conform to the following based on the mix process:

- A. For wet-mix shotcrete, the delivery equipment must be capable of delivering a steady stream of uniformly mixed material to the discharge nozzle at the proper velocity and rate of discharge. The use of positive displacement pumps equipped with hydraulic powered pistons (e.g. similar to conventional concrete piston pumps), with compressed air added at the discharge nozzle (Min 185cfm) is the preferred type of wet-mix shotcrete delivery system. Positive displacement pumps with an attached mixer will be considered.

Carefully monitor the air ring at the nozzle for any signs of blockage of individual air holes. If non-uniform discharge of shotcrete becomes apparent, stop shooting and clean the air ring or take other appropriate corrective actions

Thoroughly clean the delivery equipment at the end of each shift. Remove any build-up of coatings in the delivery hose and nozzle. Regularly inspect and clean the air ring and nozzle, and replace if required.

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Provide auxiliary shotcrete equipment such as delivery hoses, water hose, water booster pumps, blowpipes and couplings as required to complete the work.

Provide a supply of clean, dry air adequate for maintaining sufficient nozzle velocity for all parts of the work and for simultaneous operation of a blow pipe in accordance with ACI 506R-05. The air supply system must contain a moisture and oil trap to prevent contamination of the shotcrete.

- B. For dry-mix shotcrete, the delivery equipment must be capable of delivering an uninterrupted flow of pre-dampened material to the discharge nozzle at sufficient velocity in accordance with ACI 506R-05. The use of a rotary gun or pressure vessel with pre-dampening equipment is required.

Equip the discharge nozzle with a manually operated water injection system (water ring) for directing an even distribution of water through the aggregate-cement mixture. The water valve shall be capable of ready adjustment to vary the quantity of water and shall be convenient to the nozzleman. Provide water pressure at the discharge nozzle sufficiently greater than the operating air pressure to ensure that the water is intimately mixed with the other materials. If the line water pressure is inadequate, introduce a water pump into the line. Ensure that the water pressure is steady (non-pulsating).

Thoroughly clean the delivery equipment at the end of each shift. Remove any build-up of coatings in the delivery hose and nozzle. Regularly inspect and clean the water ring and nozzle lines, and replace if required.

Provide auxiliary shotcrete equipment such as delivery hoses, water hose, water booster pumps, blowpipes and couplings as required to complete the work.

Provide a supply of clean, dry air adequate for maintaining sufficient nozzle velocity for all parts of the work and for simultaneous operation of a blow pipe in accordance with ACI 506R-05. The air supply system must contain a moisture and

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oil trap to prevent contamination of the shotcrete.

- (7) Safety requirements – Implement a health, safety and environmental protection program, which conforms to the requirements of PENNDOT, OSHA and any other authorities having jurisdiction. Such programs include, but are not limited to the following:
- Ensure that all equipment, scaffolding, shoring, bracing and other devices used on the project meet the requirements of the authorities having jurisdiction.
 - Put in place a mandatory health and safety-training program for all workers, inspectors and other persons entering the workplace.
 - Protect all workers and other personnel from applied shotcrete and rebound during the shotcrete application process. As a minimum, all workers and personnel in active shotcreting areas shall wear appropriate respiratory protection devices as well as appropriate clothing and other protection equipment (hard hats, eye protection, safety boots and reflective vests). Provide eye wash equipment at shotcrete site(s).
 - Control and dispose of waste shotcrete, shotcrete trimmings from finishing processes, rebound, overspray and cementitious slurry from cleaning operations, in an approved disposal site. Take particular care to not allow any such materials to enter and/or block drainage systems. Do not allow discharge of any such materials into the surrounding waterways.
- (8) Finish - Provide undisturbed final layer of shotcrete as applied from nozzle without hand finishing, unless otherwise specified.
- (9) Curing – On completion of finishing, prevent shotcrete from drying out by moist curing using fogging or wetting or maintenance of a minimum 95% relative humidity in the area surrounding the shotcrete. Keep shotcrete continuously wet for a minimum of seven days. Wet curing is to be accomplished using one or more of the following procedures:

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- Install sprinklers, soaker hoses or other devices, which keep the shotcrete continuously wet. Avoid the use of intermittent wetting procedures, which allow shotcrete to undergo cycles of wetting and drying during the curing process.
- Absorptive mat or fabric, sand, or other covering kept continuously wet
- Curing compounds that are used as alternatives to water curing shall meet the requirements of Section 711.2(a) of Publication 408. On natural gun or flash finishes, use the application rate of 100 ft²/gal.. Do not use curing compounds on any surfaces against which additional shotcrete or other cementitious finishing materials are to be bonded unless positive measures, such as sandblasting, are taken to completely remove curing compounds prior to the application of such additional materials. Curing compounds being considered must be low odor, water-based, solvent free, and must be tested and accepted by the Department prior to use.

Continue curing until the required strength is obtained. During the curing period, maintain shotcrete above 40° F and in a moist condition as specified.

- (10) Construction testing – Cut cores from the structure and test in accordance with AASHTO T 24. Three cores from 1,000 ft² of facing or per shift are required.

Alternatively, shoot one 48 in. x 48 in. construction test panel for each nozzleman once per day. Shoot the panel in the same position and thickness as the work being done. Also shoot another test panel per nozzleman if equipment is cleaned out during the days operation. Shoot panels during the course of the work by the contractor's regular nozzleman. Field cure panels in the same manner as the work; however, the test specimens shall be soaked in water for a minimum of 40 hours prior to testing. Cut a minimum of three cores from each panel for testing in accordance with AASHTO T 24. The average compressive strength of each core of a set of three cores shall equal or exceed 85 percent of the compressive strength specified.

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- (f) Temporary shotcrete facing - Provide for approval, materials, methods, and control procedures.
- (g) Grout - Provide a neat cement grout to be used in soil nail anchorage consisting of a pumpable mixture capable of reaching a cube strength of 3 ksi in accordance with AASHTO T 106. Chemical additives that can control, bleed, or retard set in the grout are to be used only when approved in writing.
- (h) Fasteners and attachment devices - Provide high-strength nuts conforming to Section 1105.02(c)2.a of Publication 408. Provide plates and shims conforming to Section 1105.02(a)2 of Publication 408. Provide plastic centralizers of a minimum diameter 1/2 in. smaller than the nominal diameter of the drill hole.
- (i) Horizontal drains - Provide as required and shown on the plans slotted and unslotted PVC pipe conforming to AASHTO M 278. Install to the depths directed by the engineer, which will not exceed the maximum depths shown on the plans. Insure that the hole does not collapse prior to the insertion of the slotted drain. Only the front 12 in. of drain pipe shall be unslotted.
- (j) Wall drains - Provide as required and shown on the plans, prefabricated, fully wrapped, preformed drains. The core, not less than 1/4 in. thick or more than 1/2 in. thick, shall be either a preformed grid of embossed plastic or a system of plastic pillars and interconnections forming a semirigid mat. When covered with filter fabric, the core material shall be capable of maintaining a drainage void for the entire height of permeable liner. Provide a polypropylene geotextile having a minimum weight of 6 oz/yd² as the filter fabric.

1.12.4 Construction

- (a) Submittals - Provide two sets of design drawings, calculations, material certificates, construction procedures, and detailed construction sequencing plans, including excavation sequence for approval. Provide sufficient details in the design drawings to eliminate a need for shop drawings. For temporary shotcrete facing, provide complete specifications for materials and methods.

Assume all risks for work performed without approved plans.

- (b) Qualifications - Submit proof of two projects on which contractor has designed and/or installed soil nails or ground anchors in the past two years. The contractor's staff on this project is to include a

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supervising engineer with at least three years of experience in the design and construction of anchored walls.

Drilling operators and foreman are to have a minimum of two years experience installing soil nails or permanent ground anchors with the contractor's organization. Submit documentation that project personnel have appropriate qualifications. Inadequate proof of personnel qualifications shall be cause for withholding wall design approval. Changes to previously approved personnel must be approved in writing.

The shotcrete crew foreman and nozzle men must meet the following requirements:

- Furnish proof that the shotcrete crew foreman has at least three years experience in shotcrete work on projects of similar size and character along with three references from persons who were responsible for supervision of these projects. Include name, address and telephone number of references that will testify to a successful completion of these projects by the shotcrete crew foreman.
 - Furnish proof that the nozzle men are certified by the American Concrete Institute (ACI) for application of shotcrete for both vertical and overhead surfaces, using the wet-mix shotcrete process. If the Contractor elects to complete the work using the dry-mix shotcrete process, then furnish proof that the nozzle men are certified by the ACI for application of shotcrete by the dry-mix process for both vertical and overhead surface.
 - Furnish proof that the nozzle men have successfully completed three projects of similar size and character along with three references from persons responsible for the supervision of these projects. The nozzle men will have to also pass a preconstruction test, demonstrating their ability to satisfactorily place the structural elements required for this project.
- (c) Excavation – In conformance with Section 203 of Publication 408, and to the limits and construction stages indicated.
- (d) Shotcreting – After each stage cut, and in anticipation of shotcreting, clean surfaces of all loose material, mud, rebound from previously placed shotcrete, and other foreign matter that will prevent bond of

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shotcrete. Use weep holes, drain pipes, or other methods to control seepage. Where used, provide a weep hole, a 2 ft. long, 2 in. diameter, slotted drain pipe (Schedule 40 PVC) placed in pre-drilled holes sloped 5 percent to drain. Protect against contamination during shotcreting to ensure proper functioning. Saturate the prepared substrate with clean water the day before shotcrete application. Bring wetted surfaces to a saturated surface dry (SSD) condition prior to application of shotcrete. If necessary, use a blowpipe to facilitate removal of free surface water. Only oil-free compressed air is to be used in the blowpipe. Do not apply shotcrete to a dry surface or to a surface with free surface water.

All areas prepared for shotcrete application are to be reviewed and approved by the Engineer prior to any application of shotcrete.

Install permanent drainage as specified.

Apply shotcrete with the same equipment and the same technique as those used in the approved test panels. Nozzle operators performing the test panels are to be the same operators used to place shotcrete in the work. The following are provided to ensure the proper controls for alignment and cover:

- (1) Implement alignment control to establish control over line and grade and ensure that the minimum specified shotcrete thickness and cover to reinforcing steel are maintained. Verify that reinforcing steel is fixed to provide the specified cover, prior to application of any shotcrete.
- (2) Accomplish alignment control by means of devices such as shooting wires, guidestrips, depth gauges or forms. Submit the proposed means of alignment control to the Engineer for review and approval. Supports for alignment control devices shall be installed on 5 ft. centers in each direction. The supports shall be non-corrosive and designed to prevent infiltration of water through the shotcrete.
- (3) When shooting wires (also called guide wires) are used, they are to consist of a high strength steel wire kept taut during shotcreting. Remove shooting wires after completion of shotcreting operations.
- (4) Guide strips and forms are to be of such dimensions and installation configuration that they do not impede the ability of the nozzle operators to produce uniform, dense, properly

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consolidated shotcrete. In particular, installations, which are conducive to the entrapment of rebound or formation of sand pockets and voids, are not be used.

When a layer of shotcrete is to be covered by a succeeding layer at a later time, it shall first be allowed to develop its initial set. Then remove all laitance and loose material, and rebound by brooming or scraping. Remove laitance that has been allowed to take final set by sandblasting, and thoroughly clean surface.

Do not shotcrete if ambient temperature is less than 50° F and the substrate to which the shotcrete is applied is above 40° F. Maintain curing temperature as specified under Section 1.12.3(e)(9) of these specifications.

Firmly position the wire fabric to prevent vibration while the shotcrete is being applied. Lap mesh 1 1/2 squares in both directions. Bend tie wires flat in the plane of the mesh and do not form large knots. Provide a minimum cover of 2 in. of shotcrete.

Control thickness, method of support, air pressure, and water content of shotcrete to preclude sagging or sloughing off.

Fill first horizontal and vertical corners and any area where rebound cannot escape or be blown free.

Hold nozzle at a distance and angle that will place the material behind the reinforcement before any material is allowed to accumulate on its face. In the dry mix process, additional water may be added to the mix when encasing reinforcement, to facilitate a smooth flow of material behind the bars. Do not place shotcrete through more than one layer of reinforcing steel rods or mesh in one application unless preconstruction tests have demonstrated that steel is properly encased.

Construct construction joints to a 45° tapered edge. Cut plastic shotcrete with a trowel or other suitable tool to form a construction joint. Green cut with a 4350 psi water pressure jet the next day, if necessary to remove loose material. Do not featheredge (produce long tapered) construction joints and thoroughly wet before placing an adjacent section.

Repair surface defects as soon as possible after initial placement of the shotcrete. All shotcrete that lacks uniformity, that exhibits segregation, honeycombing, or lamination, or that contains any dry patches, slugs,

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voids, or sand pockets shall be removed and replaced with fresh shotcrete.

Do not repair core holes with shotcrete. Instead, fill solid with patching mortar after cleaning and thoroughly dampening.

Use the following precautions during shotcreting:

- (1) If the prevailing ambient conditions (relative humidity, wind speed, air temperature) are such that the shotcrete develops plastic shrinkage and/or early drying shrinkage cracking, terminate shotcrete application. Adopt corrective measure such as installation of wind barrier or fogging devices to protect the work, before restarting shotcrete application. Do not proceed with shotcrete application if the rate of evaporation at the shotcrete surface exceeds 2 lbs/yd²/hr as detailed in ACI 305.
 - (2) Do not use rebound or previously expended material in the shotcrete mix.
 - (3) The area to which shotcrete is to be applied shall be clean and free of rebound or overspray.
 - (4) Discontinue shotcrete application if the ambient temperature rises above 100°F, unless the Contractor adopts special hot-weather shotcreting procedures which are approved by the Engineer.
 - (5) Discontinue shotcreting when temperature drops below 50° F or when shotcrete cannot be protected.
 - (6) Maintain the air temperature at the shotcrete surfaces at 50°F or greater for at least four days after application of shotcrete. The means of maintaining the air temperature shall be approved by the Engineer. The use of unvented heaters, which give rise to carbonation, is prohibited.
- (e) Nail installation - Drill holes for soil nails at the location shown. Provide the soil nail length necessary to develop adequate load capacity to satisfy testing acceptance criteria for the design load required, but not less than the length shown on approved plans. Casing may be necessary to maintain a clean open hole drilled to the size and inclination shown. Drilling methods and grouting pressure are at the option of the contractor. At the point of entry, the nail angle shall be within plus or minus 3 degrees of that shown on the approved plans.

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Subsidence or physical damage by such operations shall be cause for immediate cessation of operations and repair at the contractor's expense.

Inject grout at the lowest point of the drill hole. Pump grout through grout tubes, casing, hollow-stem augers, and drill rods until the hole is filled to prevent air voids. Fill with grout progressively from the bottom to top. Provide grouting equipment capable of continuous mixing and producing a grout free of lumps. Place a nail in each drilled hole within 15 minutes of the grout injection.

Place centralizers at 10 ft. intervals in total length, and ensure that no less than 1.5 in. of grout cover is achieved at all locations along the tendon.

Lightly stress installed nails to take up any slack after the grout has reached a compressive strength of at least 1.5 ksi.

(f) Construction sequencing - Follow the construction sequence on the approved plans closely.

(g) Nail testing:

(1) Equipment - Provide a dial gauge capable of measuring to 0.001 in. to measure movement. A hydraulic jack and gauge calibrated as a unit shall be used to apply the test load. Provide pressure gauge graduated in 100 psi increments or less and use to measure the applied load. Apply test load incrementally.

(2) Pullout testing - Install one nail per horizontal row, but no more than 3 percent of the total number of nails as non-service nails, and load test to pullout failure (maintained movement without increased load). Install and test at each level at a rate consistent with construction operations. Choose test length of nail to cause pullout failure prior to steel yield, but not at less than 8 ft.. Provide a minimum ungrouted zone 3 ft. long to the face. The method of installation and size of the drill hole shall be the same as for the production nails.

Grout in place each test nail as part of a regular production grouting process. After grouting, do not load for a minimum of three days.

Perform pullout test by incrementally loading the nail in accordance with the following schedule. Measure nail movement and record to the nearest 0.001 in. with respect to an independent fixed reference point at each increment of load.

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Monitor the test with a pressure gauge. The load hold period shall start as soon as the test load is applied. Movement shall be recorded at 1 minute, 2, 3, 4, 5, 6 and 10 minutes. Each increment of load shall be no greater than 15 percent of the steel yield strength of the nail. Terminate loading at failure or earlier, at the option of the contractor, if the design friction limit is demonstrated.

- (3) Acceptance criteria - The nail is acceptable if the developed friction limit at failure is greater than the design friction limit. Unacceptable test results shall result in modification to design and/or construction procedures. Any modification of design or construction procedures shall be at no change in the contract prices. Graphs shall be plotted during the test of deflection against load.
- (h) Cast-in-place concrete facing - At the completion of the sequenced construction and where required, construct cast-in-place structural facing in accordance with the provisions of Section 1001, Pub. 408.
- (i) Precast concrete facing - At the completion of the sequenced construction and where required, construct precast concrete structural facing in accordance with the provisions for precast concrete panels for mechanically stabilized embankments. Attachment devices are to be shown on the plans as to size and material composition.

1.12.5 Measurement and Payment

Lump sum.

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VOLUME 2

APPENDIX P – JACKING AND SUPPORTING THE SUPERSTRUCTURE

The following sections are excerpts from the AASHTO Maintenance Manual for Roadways and Bridges 2007 and are provided for guidance in developing shoring of superstructures.

Jacking and Supporting the Superstructure

Bridge rehabilitation and superstructure repair often require jacking to provide load transfer and bridge support while repairs are made. If jacking is required, both the safety of any vehicular traffic continuing to use the bridge while maintenance and repair are conducted and the safety of all working personnel must be considered.

- Jacks and jacking supports must be straight, plumb, and of sufficient capacity to support the portion of the bridge being lifted.
- Place jacks at points that will not damage the structure; reinforce jacking points on the structure if necessary.
- Before jacking, check deck joints for offsets that might be damaged from differential movement between spans during jacking; to prevent damage, check railings and disconnect them, if needed.
- Uniformly raise and lower jacks to distribute the jacking load evenly and prevent overstressing or twisting the bridge.
- Position blocks adjacent to the jacks to increase their height as the structure is raised, minimizing any loss of support if a jack fails during the jacking operation.
- Do not permit traffic on the bridge while it is supported by jacks. If traffic is permitted on the bridge while it is supported by blocks, a vertical transition slope should be provided to avoid abrupt changes in the road surface, to provide a safe riding surface, to prevent damage to the bridge, and to minimize any vertical acceleration loading from the traffic.

Jacking the Superstructure. The procedure should be designed by a Professional Engineer licensed in the Commonwealth of Pennsylvania for each jacking set-up, ensuring that it is adequate for the job to be undertaken. This design and review must account for the following factors:

- Dead-load reaction to bear on the jacks.
- If the bridge cannot be closed to traffic during jacking (preferable), then the expected live load on the jacks must be included.
- The size and number of jacks required.
- The location of the jacks.
- Any temporary bents or cribbing required to support the jacks.
- Any modifications to bridge structural members required at the jacking points so that the bridge members can sustain the jacking pressure.
- Sufficient space at deck joints to permit differential movement between spans.
- Defining the height to which the structure needs to be jacked, jacking only as high as is absolutely necessary to conduct the required maintenance.

Generally, the steps and precautions included within a jacking procedure to ensure successful, safe operation are as follows:

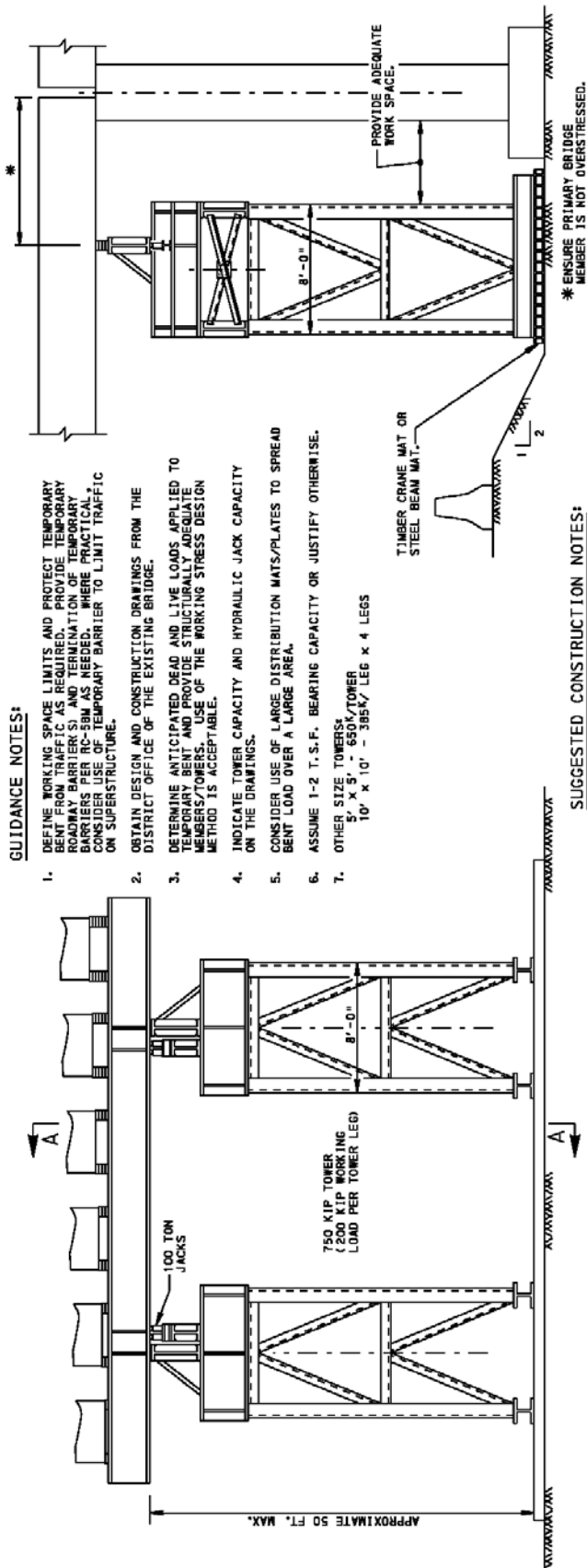
1. Construct the necessary bents and cribbing to support the jacks when it is not possible to locate supports on the existing substructure. An adequate foundation to prevent differential settlement is very important.
2. As necessary, reinforce bridge members to withstand the force of the jacks.

3. If necessary, disconnect railing and utilities.
4. Place jacks snugly in position.
5. Restrict vehicular traffic on the span while it is supported by jacks.
6. Raise the span by jacking. Pressure gauges should be used to ensure that all of the jacks are lifting the span evenly.
7. Use observers placed at strategic points to watch for signs of structural distress because of jacking.
8. Jack, block, and rejack until the required position is achieved.
9. Protect joints and provide a transition to the span with steel plates if traffic is maintained while the span is on blocks.
10. Check periodically to ensure that there is no differential settlement.
11. After the repairs are completed, remove the blocks using the jacks.
12. After the span has been lowered into place, ensure that the deck joints are functioning properly, that the alignment has not been changed, that there is adequate space for expansion without debris or restriction in the joints, and that the joint seal is watertight.

The following schematics are provided for guidance in developing shoring plans:

- Partially Supported Superstructure
- Fully Supported Superstructure
- Partially Supported Superstructure Alternate Shoring Method

DMA TEMP BRACING.DWG



ELEVATION

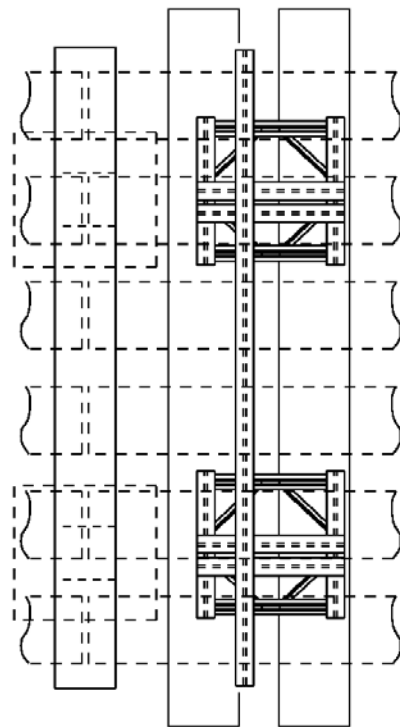
SECTION A-A

GUIDANCE NOTES:

1. DEFINE WORKING SPACE LIMITS AND PROTECT TEMPORARY STRUCTURES FROM COLLISIONS. PROVIDE TEMPORARY ROADWAY BARRIERS, AND TERMINATION OF TEMPORARY BARRIERS PER RC-50M AS NEEDED. WHERE PRACTICAL, CONSIDER USE OF TEMPORARY BARRIER TO LIMIT TRAFFIC ON SUPERSTRUCTURE.
2. OBTAIN DESIGN AND CONSTRUCTION DRAWINGS FROM THE DISTRICT OFFICE OF THE EXISTING BRIDGE.
3. DETERMINE ANTICIPATED DEAD AND LIVE LOADS APPLIED TO TEMPORARY BENT AND PROVIDE STRUCTURALLY ADEQUATE MEMBERS/TOWERS. USE OF THE WORKING STRESS DESIGN METHOD IS ACCEPTABLE.
4. INDICATE TOWER CAPACITY AND HYDRAULIC JACK CAPACITY ON THE DRAWINGS.
5. CONSIDER USE OF LARGE DISTRIBUTION MATS/PLATES TO SPREAD BENT LOAD OVER A LARGE AREA.
6. ASSUME 1-2 T.S.F. BEARING CAPACITY OR JUSTIFY OTHERWISE.
7. OTHER SIZE TOWERS:
5' x 5' - 650K/TOWER
10' x 10' - 385K/LEG x 4 LEGS

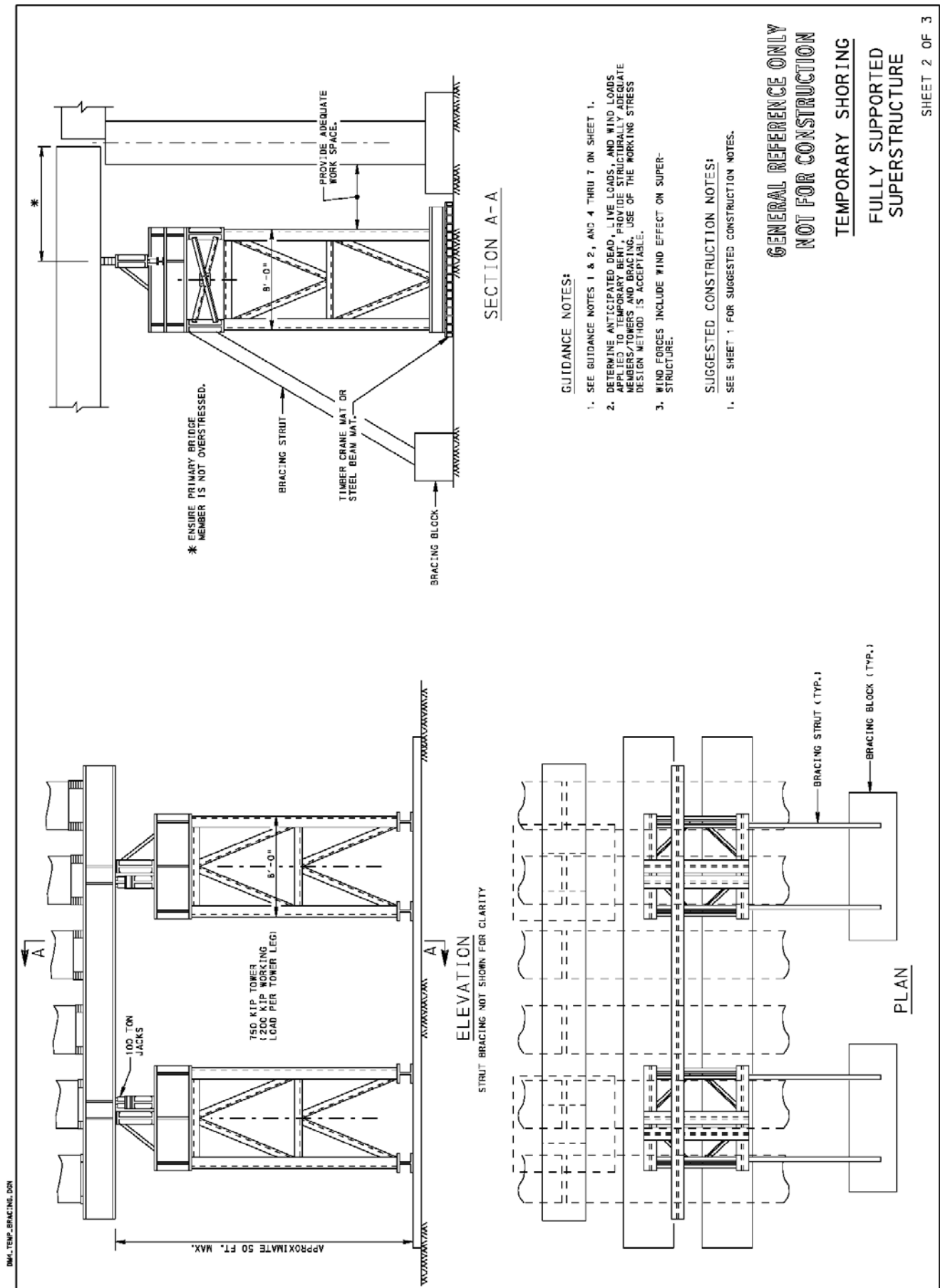
SUGGESTED CONSTRUCTION NOTES:

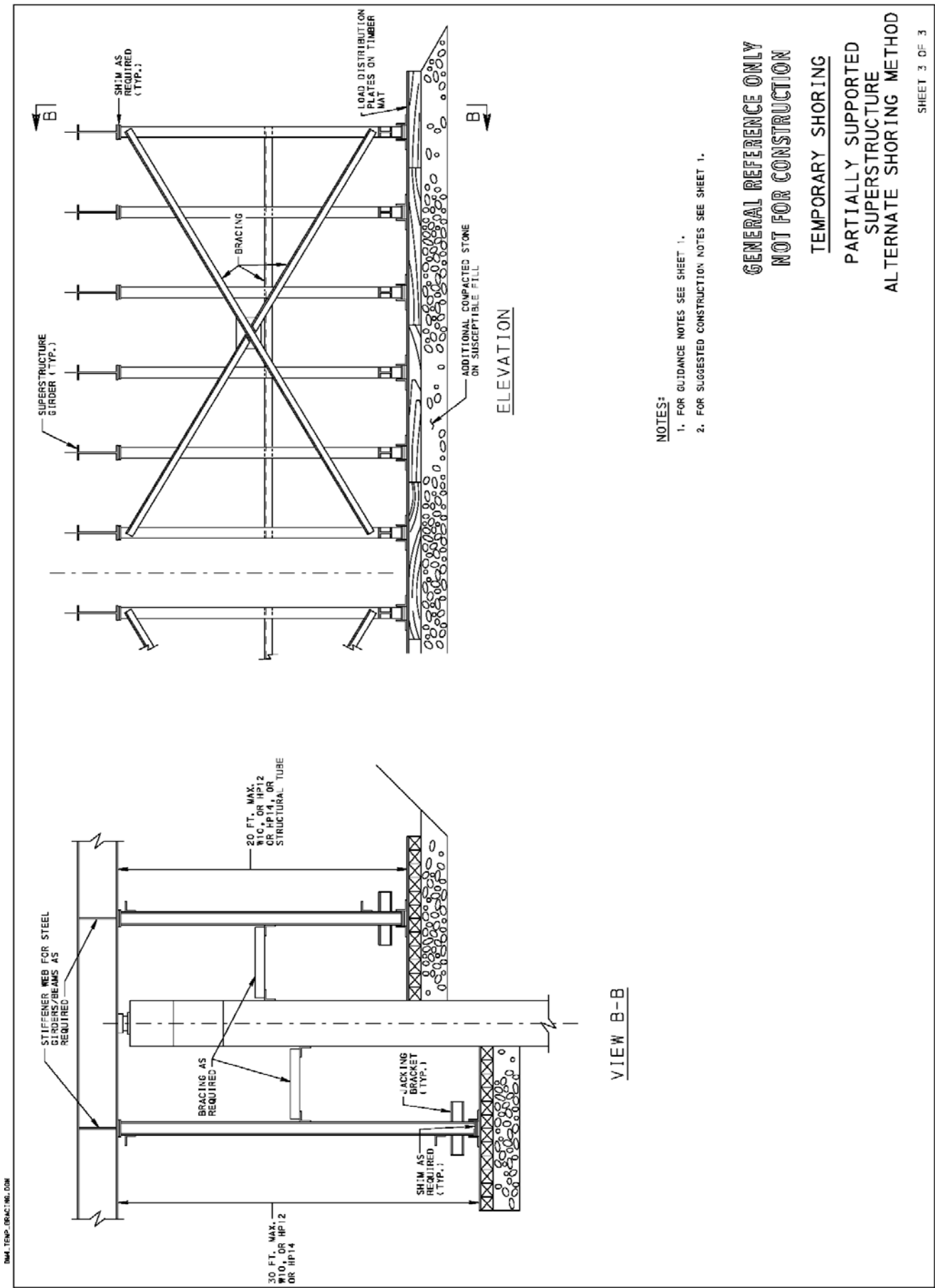
1. PROVIDE TEMPORARY BENT AND BRACING AS REQUIRED TO MAINTAIN STRUCTURAL STABILITY.
2. PROVIDE MATERIALS AND WORKMANSHIP IN ACCORDANCE WITH PUBLICATION 408.
3. EVALUATE IN-SITU FOUNDATION MATERIAL AND OVER EXCAVATE ANY UNSUITABLE FOUNDATION MATERIAL AND BACK FILL WITH COMPACTED NO. 2A COARSE AGGREGATE WHEN REQUIRED.
4. DIVERT SURFACE WATER RUNOFF AWAY FROM THE TEMPORARY BENT FOUNDATION.
5. PROVIDE SHIMS AND BRACING AS REQUIRED TO MAINTAIN STRUCTURAL STABILITY AND SUPERSTRUCTURE ALIGNMENT. BENT COMPONENTS ESPECIALLY SHIMS AND POINT OF EXISTING SUPERSTRUCTURE SUPPORT SHALL BE TACKED/BOLTED/GUIDED TO PREVENT UNDESIRABLE STRUCTURAL MOVEMENTS TRANSFERRED TO THE TEMPORARY BENT.
6. PROVIDE TEMPORARY BRACING CALCULATIONS AND DRAWINGS SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN PENNSYLVANIA.
7. DO NOT CAUSE UNACCEPTABLE OVERSTRESS TO OTHER BRIDGE COMPONENTS.
8. CERTIFY FALSE WORK PER PUB. 408, SECTION 105.03 (c).
9. JACK SIMULTANEOUSLY WHEREVER POSSIBLE. ALTERNATELY JACK WITH ASSIST POINT.



PLAN

GENERAL REFERENCE ONLY
NOT FOR CONSTRUCTION
TEMPORARY SHORING
PARTIALLY SUPPORTED
SUPERSTRUCTURE





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