

BRIDGE DESIGN GUIDE



August 2003

Maine Department of Transportation



BRIDGE DESIGN GUIDE



Cover photos (clockwise from top): Mill Bridge, Alton-Old Town; Presumpscot Falls Bridge, Falmouth; Lows Covered Bridge, Guilford-Sangerville; and Middle Branch Bridge, T05 R09 (Ebeemee)

Prepared by **Guertin Elkerton & Associates**



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TABLE OF CONTENTS

Chapter 1 – General

Chapter 2 – Preliminary Design

Chapter 3 – Loads

Chapter 4 – Superstructures

Chapter 5 – Substructures

Chapter 6 – Concrete

Chapter 7 – Steel

Chapter 8 – Buried Structures

Chapter 9 – Timber/Engineered Wood Composites

Chapter 10 – Rehabilitation

Appendix A – Bridge Nomenclature

Appendix B – PDR Forms

Appendix C – Hydrology/Hydraulics

Appendix D – Standard Notes

Chapter 1

GENERAL



Deer Isle-Sedgwick Bridge, Deer Isle-Sedgwick



Bailey Island Bridge, Harpswell

1	GENERAL.....	1-1
1.1	Introduction.....	1-1
1.2	General Team Approach Guidelines	1-3
1.3	Final Design Issues	1-3
1.3.1	Plans, Specification and Estimate (PS&E)	1-3
1.3.1.1	Plans	1-3
1.3.1.2	Structural Design Computations.....	1-3
1.3.1.3	Geotechnical Design Computations	1-4
1.3.1.4	Bridge Ratings.....	1-4
1.3.1.5	Special Provisions	1-4
1.3.1.6	Engineer's Estimate	1-4
1.3.1.7	Bridge Information Form.....	1-7
1.3.1.8	Budgetary Information	1-7
1.3.2	Maintenance of Traffic.....	1-7
1.3.3	Survey Layout	1-7
1.4	Design Check Guidelines	1-7
1.5	Small Bridge Initiative	1-11
1.5.1	Field Survey Considerations.....	1-11
1.5.2	Right-of-Way Considerations.....	1-11
1.5.3	Geotechnical Considerations.....	1-11
1.5.4	Hydrologic Considerations.....	1-12
1.5.5	Minimization of Approach Work.....	1-12
1.5.6	Reduction of Structural Design Effort	1-12
1.5.7	Contracting Strategies	1-13
1.6	Non-Vehicular Bridges.....	1-14
1.7	Aesthetics	1-15
1.7.1	General.....	1-15
1.7.2	Design Considerations	1-15
1.7.2.1	Superstructure.....	1-16
1.7.2.2	Substructure	1-16
1.7.2.3	Color.....	1-21
	References	1-24
	Table 1-1 Type/Material Selection Guide for Small Bridge Projects.....	1-13
	Figure 1-1 Consistent Use of Flares	1-17
	Figure 1-2 Methods to Thin Appearance of Fascia	1-18
	Figure 1-3 Effect of Overhang Length on Beam Shadow.....	1-19
	Figure 1-4 Hammerhead Pier Proportions	1-19
	Figure 1-5 Variations of Cantilever Length and Batter	1-20
	Figure 1-6 Ratio of Pier Width to Fascia Depth.....	1-22
	Figure 1-7 Effect of Column Shape on Shadows and Thin Appearance	1-23

1 GENERAL

1.1 Introduction

This document is intended to provide guidance to those performing design for the Bridge Program of the Maine Department of Transportation (MaineDOT). It should provide clarity to the design thought process, and serves as a supplement to the applicable AASHTO standards. It should be used in conjunction with good engineering judgment.

This document is a companion volume to the Bridge Program's "Project Management Guide" and "Bridge Plan Development Guide."

The Mission and Goals of the Bridge Program are on the following page.

Mission

The Bridge Program delivers safe, cost effective, quality bridge projects to our customers on schedule.

Goals and Objectives

1. Reduce backlog of deficient bridges

- o Comprehensive planning effort to prioritize bridge needs
- o Resources to deliver the Bridge Program

2. Ensure project timeliness

- o Complete construction on schedule
- o Meet project schedule needs

3. Assure project quality and cost effectiveness

- o Provide quality projects that meet the purpose and need at optimum cost
- o Improve staff effectiveness through continuous employee development

4. Foster public satisfaction

- o Share information, seek public input, and build public trust

1.2 General Team Approach Guidelines

The Bridge Program is regionally organized into Self-Directed Work Teams (SDWTs), each led by a Project Manager. In addition to the Project Managers, each team is composed of Structural Designers, Design Technicians, a Geotechnical Designer, Construction Residents, Construction Inspectors, a Utility Coordinator, a Mapper, an Appraiser, and a Team Coordinator. The environmental coordination function is managed by the Environmental Coordinator from MaineDOT's Environmental Office, while survey functions are managed by the regional Survey Coordinator.

Each team member has a specific role that is integral to the success of the project as it moves through the project development process. The Structural Designer and the Geotechnical Designer provide the design expertise, and use the resources of the team to provide input into the decision-making that is part of every design.

1.3 Final Design Issues

1.3.1 Plans, Specification and Estimate (PS&E)

This documentation includes a package of information that is used to prepare the bid documents for advertising a project. The package is prepared by the project team and further assembled by the Contracts Technician within the Program. It includes the following items, with the responsibility of the Designers noted:

1.3.1.1 Plans

The plans consist of complete contract drawings that adequately display the design with enough detail to construct the project. The plans are the responsibility of the Design Technician, but must be reviewed by the Designers for conformance to the design. During the development of the plans, communication is essential to avoid rework. Standard notes are found in Appendix D. Plan layouts and detailing practices can be found in the Bridge Program's "Bridge Plan Development Guide."

1.3.1.2 Structural Design Computations

Detailed design computations from the selected alternate are bound, dated, and submitted by the Structural Designer as part of the PS&E package. Design computations should include all references and assumptions used during design. After submission, they are retained in the Computations file cabinet of the Bridge Program.

1.3.1.3 Geotechnical Design Computations

Geotechnical design computations are included as an appendix of the Geotechnical Design Report. Design computations include all references and assumptions used during design. After completion of the project, the geotechnical file is retained in the Materials, Testing, and Exploration archives in Bangor.

1.3.1.4 Bridge Ratings

Each bridge must be rated by the Structural Designer with a live load rating using the Load and Resistance Factor Rating (LRFR) method. Refer to the Manual for Condition Evaluation of and Load and Resistance Factor Rating (LRFR) of Highway Bridges, October 2003, with interims, for guidance in the live load rating calculation.

1.3.1.5 Special Provisions

In most cases, Supplemental Specifications, commonly used Special Provisions, and/or project specific Special Provisions will be necessary to complement the Standard Specifications. Current Supplemental Specifications and commonly used Special Provisions are available for review. The Designers review and format these specifications for necessary inclusion in the contract documents. If project specific specifications are warranted, the Designers write and format them for the PS&E Package. The Project Manager may be involved in writing some project specific specifications that are not design related.

1.3.1.6 Engineer's Estimate

This confidential document consists of a detailed estimate of quantities and costs necessary to construct the project. Typically, the Design Technician, with input from the Designers and Project Manager, develops the pay item list and computes the estimated quantities. The Design Technician then inputs the quantities into ESTIMATOR, which will provide automatic weighted average costs for each of the pay items. The Designers are responsible for reviewing those costs and adjusting them where needed, using engineering judgment. For a complete guide to developing an estimate or check, refer to the Bridge Program's "Bridge Plan Development Guide."

Bridge Information Form

Project

PIN _____ Bridge Number _____
 Location _____
 Bridge Name _____

Project Manager _____
 Lead Designer _____
 Lead Technician _____
 Resident _____

Design Code

LRFD LFD Other (explain) _____

Bridge Parameters

Number of Spans _____

Multiple Span Configuration _____

Number of Sidewalks _____

Bridge Length (CL Brg Abut to CL Brg Abut) _____ FT

Buried Structure Total Span Length (use clear spans) _____ FT

Skew _____ °

Bridge Width (Fascia-to-Fascia) _____ FT

Roadway Width (Curb-to-Curb or Rail-to-Rail) _____ FT

Buried Structure Barrel Length _____ FT

Beam Spacing _____ FT

Slab Thickness _____ IN

Approach Length (inc. buried structure, but exc. bridge) _____ FT

Scope

- BIKEWAY
- BRIDGE CONSTRUCTION-NEW
- BRIDGE CULVERT REHABILITATION
- BRIDGE CULVERT REPLACEMENT
- BRIDGE DECK REHABILITATION
- BRIDGE DECK REPLACEMENT
- BRIDGE IMPROVEMENT
- BRIDGE PAINTING
- BRIDGE RAIL & CURB IMPROVEMENT
- BRIDGE REHABILITATION
- BRIDGE REMOVAL
- BRIDGE REPLACEMENT
- BRIDGE SUBSTRUCTURE REHAB.
- BRIDGE SUPERSTRUCTURE REPLACE.
- BRIDGE WEARING SURFACE REPLACE.
- BRIDGE WIDENING
- TEMPORARY BRIDGE
- Other (explain)

Work Attribute

- Consultant X-LARGE
- Consultant LARGE
- Consultant MEDIUM
- Consultant SMALL
- Over Water Replace. X-LARGE
- Over Water Replace. LARGE
- Over Water Replace. MEDIUM
- Over Water Replace. SMALL
- Over Water Replace. X-SMALL
- Overpass Replace. LARGE
- Overpass Replace. MEDIUM
- Rehab X-LARGE
- Rehab LARGE
- Rehab MEDIUM
- Rehab SMALL
- Paint SIMPLE
- Paint COMPLEX
- Other (explain)

Bridge Information Form

Estimated Quantities

Volume of Abutment Concrete		CY
Volume of Pier Concrete		CY
Volume of CIP or Precast Rigid Frame Concrete		CY
Volume of Structural Slab Concrete		CY
Total Length of Concrete Beams/Girders		FT
Weight of Structural Steel		LB
Weight of Bituminous on Bridge		LB
Weight of Substructure Rebar		LB
Weight of Superstructure Rebar		LB

Buried Structure Type

- Structural Steel Pipe or Pipe Arch
- Structural Steel Plate Arch or Frame with CIP Footings
- Structural Steel Frame with Metal Footings or Bottom Plate
- Structural Aluminum Pipe or Pipe Arch
- Structural Aluminum Plate Arch or Frame with CIP Footings
- Structural Aluminum Frame with Metal Footings or Bottom Plate
- Precast Concrete Frame on Concrete Footings
- Precast Concrete Box
- Cast-in-Place Rigid Frame or Arch
- Plastic Pipe
- Other (explain) _____

Superstructure Type (Primary Load-Carrying Members)

- | | |
|---|---|
| <ul style="list-style-type: none"> <input type="checkbox"/> Steel - Rolled Beam <input type="checkbox"/> Steel - Welded Constant Depth Girder <input type="checkbox"/> Steel - Welded Haunched Girder <input type="checkbox"/> Steel - Rolled Beam and Welded Girder <input type="checkbox"/> Steel - Welded Box Girder <input type="checkbox"/> Precast Prestressed Voided Slab <input type="checkbox"/> Precast Prestressed Nonvoided Slab <input type="checkbox"/> Precast Prestressed Butted Box Beam <input type="checkbox"/> Precast Prestressed Spread Box Beam <input type="checkbox"/> Precast Prestressed New England Bulb Tee <input type="checkbox"/> Precast Prestressed AASHTO I Girder <input type="checkbox"/> CIP Concrete - Slab <input type="checkbox"/> CIP Concrete - T-Beam <input type="checkbox"/> CIP Concrete - Open Spandrel Arch <input type="checkbox"/> Post-Tensioned Concrete - Segmental Box <input type="checkbox"/> Inverset | <ul style="list-style-type: none"> <input type="checkbox"/> Suspension <input type="checkbox"/> Cable-Stayed <input type="checkbox"/> Steel - Through Truss <input type="checkbox"/> Steel - Pony Truss <input type="checkbox"/> Steel - Deck Truss <input type="checkbox"/> Timber - Through Truss <input type="checkbox"/> Timber - Pony Truss <input type="checkbox"/> Timber - Deck Truss <input type="checkbox"/> Timber - Covered <input type="checkbox"/> Timber - Solid Sawn Beam <input type="checkbox"/> Timber - Glulam Beam <input type="checkbox"/> Timber - Glulam Direct Span <input type="checkbox"/> FRP Reinforced Glulam Beam <input type="checkbox"/> Other (explain) _____ |
|---|---|

Wearing Surface Type

- | | |
|--|--|
| <ul style="list-style-type: none"> <input type="checkbox"/> Bituminous with Membrane Waterproofing <input type="checkbox"/> Bituminous with HP Membrane Waterproofing <input type="checkbox"/> Bituminous over Fill on Buried Structure <input type="checkbox"/> Rosphalt <input type="checkbox"/> Timber | <ul style="list-style-type: none"> <input type="checkbox"/> Concrete - Integral <input type="checkbox"/> Concrete - Unreinforced <input type="checkbox"/> Concrete - Reinforced <input type="checkbox"/> Other (explain) _____ |
|--|--|

Bridge Information Form

Deck Type

- | | |
|---|---|
| <input type="checkbox"/> CIP Concrete
<input type="checkbox"/> CIP Concrete with Precast Deck Panels
<input type="checkbox"/> Precast Concrete
<input type="checkbox"/> Open Steel Grid
<input type="checkbox"/> Concrete-Filled Steel Grid
<input type="checkbox"/> Orthotropic
<input type="checkbox"/> Exodermic | <input type="checkbox"/> Timber
<input type="checkbox"/> Glulam
<input type="checkbox"/> Other (explain)
<hr/> |
|---|---|

Composite Deck Design?

-
- Yes
-
- No

Bridge Rail Type

- | | |
|--|--|
| <input type="checkbox"/> 2-Bar Steel Rail
<input type="checkbox"/> 3-Bar Steel Rail
<input type="checkbox"/> 4-Bar Steel Rail
<input type="checkbox"/> 2-Bar & 4-Bar Steel Rail
<input type="checkbox"/> Texas Classic Concrete Rail
<input type="checkbox"/> Maine Concrete Rail
<input type="checkbox"/> Concrete Barrier
<input type="checkbox"/> Concrete Barrier with Mounted Steel Rail
<input type="checkbox"/> Concrete Barrier with Mounted Aluminum Rail | <input type="checkbox"/> Bridge-Mounted Guardrail
<input type="checkbox"/> Bridge-Mounted Thrie Beam
<input type="checkbox"/> 2-Bar Aluminum Rail
<input type="checkbox"/> 4-Bar Aluminum Rail
<input type="checkbox"/> Timber and Steel Rail
<input type="checkbox"/> Timber Rail
<input type="checkbox"/> Other (explain)
<hr/> |
|--|--|

Abutment Type

-
- Stub Cantilever
-
-
- Medium Cantilever (5' < Wall < 15')
-
-
- High Cantilever (Wall > 15')
-
-
- Mass
-
-
- Integral
-
-
- Semi-Integral
-
-
- Other (explain)
-
-

Pier Type

-
- Mass
-
-
- Pile Bent
-
-
- Hammerhead
-
-
- Shaft
-
-
- Multiple Column
-
-
- Other (explain)
-
-

Abutment Foundation Type

-
- End-Bearing H-Pile
-
-
- Friction H-Pile
-
-
- End-Bearing Pipe Pile
-
-
- Friction Pipe Pile
-
-
- Rock-Socketed H-Pile
-
-
- Rock-Socketed Pipe Pile
-
-
- Spread Footing on Bedrock
-
-
- Spread Footing on Soil
-
-
- Drilled Shaft
-
-
- MSE Wall
-
-
- Other (explain)
-
-

Pier Foundation Type

-
- End-Bearing H-Pile
-
-
- Friction H-Pile
-
-
- End-Bearing Pipe Pile
-
-
- Friction Pipe Pile
-
-
- Rock-Socketed H-Pile
-
-
- Rock-Socketed Pipe Pile
-
-
- Spread Footing on Bedrock
-
-
- Spread Footing on Soil
-
-
- Drilled Shaft
-
-
- Other (explain)
-
-

Comments:

1.3.1.7 Bridge Information Form

The form preceding this section is completed by the Structural Designer as part of the PS&E package. It is available electronically as an Excel spreadsheet, and is used to establish a reliable database for tracking project features and preliminary estimate costs, and for adjusting costs in Engineer's Estimates.

1.3.1.8 Budgetary Information

In addition to the Engineer's Estimate, there are several documents that must be completed to ensure that the updated costs of the project are distributed throughout the MaineDOT. The Project Manager completes other budgetary forms, including the Project Cost Summary Form, Construction Authorization Form, and the portion of the PS&E form that pertains to costs. These forms can be found in the Project Management Guide.

1.3.2 *Maintenance of Traffic*

A Traffic Control Plan must be developed for every project. Responsibility for this plan is with either the Contractor, or MaineDOT, as determined at the PS&E stage. The complexity of the project may steer the Structural Designer toward keeping this responsibility within MaineDOT, to assure compliance with the conceptual design. Any traffic control plan must comply with the latest edition of the Manual of Uniform Traffic Control Devices (MUTCD).

1.3.3 *Survey Layout*

A DAB (describe alignment bearing) Report or similar geometric output file should be submitted by the Designer as part of the PS&E package. This file is used in conjunction with the horizontal alignment files to generate all necessary field layout information. For a more comprehensive description of required information, please refer to the "Bridge Plan Development and Estimating Guide."

Currently, MaineDOT provides Contractors with horizontal and vertical project control and quality assurance only. The Contractor is responsible for all remaining construction survey activities.

1.4 **Design Check Guidelines**

As a general rule, the design check of a structure or foundation will be assigned to a Structural Designer or a Geotechnical Designer, respectively (Design Checker). The check and/or review of the construction plans and the Engineer's

Estimate will be assigned to a Design Technician (Detail Checker or Reviewer). Design checks should be completed before any structural detailing is done whenever possible. Additional Structural/Geotechnical Designers and Design Technicians may be assigned to assist in the checking and review process for more complex projects or to facilitate project schedules. Occasionally, at the Team's discretion, the Design Checker and Detail Checker or Reviewer may be the same person.

There are six general areas where checking and/or review of a project should occur and these are:

- o Preliminary Design Reports
- o Geotechnical Design Reports (including Series 100 Reports)
- o Hydrology/Hydraulics/Scour
- o Final Structural and Approach Design of In-House Projects
- o Final Structural and Approach Design of Consultant Projects
- o Shop Drawings

The Structural or Geotechnical Designer (Designer) is responsible for a cost effective and efficient design in accordance with this "Bridge Design Guide" and the Preliminary Design Report (PDR). The Design Checker is responsible for assuring that this goal was met. The Designer is then responsible for communicating the design parameters and configuration to the Design Technician. The function of the Design Checker is not to re-design a project, but to perform the expected level of check or review as follows:

- o *Independent Design Check*: Perform an independent structural or geotechnical analysis of designed components to assure that the design criteria are met. This level of design check is appropriate for structural and geotechnical components of new and rehabilitated structures, and horizontal and vertical geometry of approaches.
- o *Design Review*: Use engineering judgment to evaluate the design of structural and geotechnical components without performing a structural analysis. This level of design review is appropriate for geotechnical reports (including Series 100 Reports), hydrology and hydraulics, consultant PDRs, consultant final designs, and structural notes.

PDRs are subject to the team process in which Coachpoint meetings and consultations with Team Members, municipalities, state and federal agencies, peers, and Functional Managers provide feedback and direction for the project. A completed PDR is reviewed and approved by the Engineer of Design for its

design recommendations, and by the Assistant Program Manager for its budget, prior to the general distribution of the PDR for comments. The hydrology, hydraulics, and scour for a project should undergo a design review.

When a design is being performed by a new or inexperienced Structural Designer, the Design Checker should be an experienced Structural Designer. Inexperienced Structural Designers may be assigned as the Design Checker for designs done by experienced Structural Designers. All Geotechnical Reports should be checked and reviewed by experienced Geotechnical Designers only.

The Design Technician is responsible for developing good quality construction plans that will accurately communicate the Designer's vision to the Construction team members and to the Contractor. The Detail Checker or Reviewer is responsible for assuring that this goal was met. The function of the Detail Checker or Reviewer is not to re-detail a project, but to perform the expected level of check or review as follows:

- o *Significant Detail Check:* Verify significant details of major components and review completeness of plans (are there adequate sections, plan views, elevations, etc.). This level of detail check is appropriate for such items as approach plans, structural details of new and rehabilitated structures, foundation details, boring sheets, and estimated quantities.
- o *Detail Review:* Use engineering judgment to evaluate the details without performing verification calculations, and review completeness of plans. This level of detail review is appropriate for such items as wearing surface projects, structural plate projects, reinforcement schedules, pay item lists, general notes, and consultant final plans.

The quality of a project begins with the Structural Designer, Geotechnical Designer, and the Design Technician. It is their responsibility to produce the preferred level of accuracy and completeness. They should not rely on the Project Checkers and/or Reviewers to fill in the missing pieces.

The Checkers and/or Reviewers should be aware of any commitments to town officials or other agencies to assure that they have been incorporated into the design of the project. The Design Checker should note all the changes that he/she feels are necessary for the Designer's consideration. The Design Checker may also point out where the Designer could have used better judgment in design concepts, structural features, or structural economy. At times, a poor practice employed by the Designer may be allowed to stand in order to expedite the project. However, such poor practices, even if they are not of great consequence, should be pointed out to the Designer for his/her own benefit in order to prevent future repetition of that poor practice.

The Detail Checker should note all the changes that he/she feels are necessary for the Design Technician's consideration, if such changes may result in a significant cost reduction impact or if there is a risk of construction error. The Detail Checker should recommend a plan layout change only if there is a risk of construction error. If construction plans are poorly organized and difficult to decipher, then the Detail Checker should bring this to the Design Technician's attention for future reference. After the check/review process is completed, the Designer should inform the Detail Checkers of any additional changes made to the construction plans as a result of comments received from other programs, agencies, or the Engineer of Design.

When plans have been developed by new or inexperienced personnel, the Detail Checker or Reviewer should be an experienced Design Technician, Structural Designer, or Resident. The level and extent of detail check should be increased accordingly, due to the increased potential for omissions and errors. Inexperienced Design Technicians may be assigned as Detail Checker or Reviewer on plans developed by experienced Design Technicians.

If a dispute occurs, the disputants (whether they are the Design Checker and the Designer, or the Detail Checker and the Design Technician) should attempt to resolve the dispute themselves, consulting with their peers as the need arises. If an agreement cannot be reached even after consultation with their peers, then the case should be presented to an arbiter appointed by the Engineer of Design.

This same procedure applies if there is a disagreement between the Designer and the Design Technician. Past practice has been that the Designer has final say on the project's plans. Designers and Design Technicians should respect each other's professional skills and knowledge in their areas of expertise.

A 2% to 5%+ margin of error is acceptable for design overstress for either the superstructure or substructure design. A 10% to 15%+ margin of error is acceptable for design understress before a design reduction is recommended. These percentages depend greatly on the cost impacts and on the uncertainty of the design assumptions. For example, if the Structural Designer proposes to use #6 bars at 6" and the Design Checker finds that this is 20% oversized and that #5's and #6's alternating at 6" will probably work, the overdesign may be preferred for its simplicity in rebar detailing, ordering, and placement.

Margins of error for dimensions of significant details vary depending upon the structure component and type. For structural steel, the margin of error may be from 1/8" to 1/2". For camber dimensions, the margin may be from 1/8" to 1/4". Blocking dimensions should be within 0.02 feet. A 1/4" to 1/2" margin of error is acceptable for cast-in-place concrete and a 1/8" to 1/4" margin of error is acceptable for precast, prestressed concrete. For cast-in-place concrete substructures, the nearest 1/2" is acceptable.

1.5 Small Bridge Initiative

A reduced project delivery process should be considered for any bridge project with a structure of 50 foot span or less.

These small bridge projects may not need a full hydrologic analysis, complete topographical field survey, right-of-way takings, utility adjustments, public meetings, subsurface investigations, or other activities typically used for larger projects. If a reduced process is considered, the project team should conduct a site review to determine the degree of effort and the scope of work. Discussions should also take place with abutting property owners and municipal officials.

1.5.1 Field Survey Considerations

Project characteristics that favor limited or no survey include:

- o Rural setting with few manmade features near the bridge
- o No permanent right-of-way acquisitions
- o In-kind structure replacement with very limited approach work
- o Acceptable existing roadway geometry
- o No sensitive environmental resources needing to be mapped
- o Lack of critical cross sectional issues

1.5.2 Right-of-Way Considerations

If practical, project limits and scope can be adjusted to require only work permits or construction easements.

1.5.3 Geotechnical Considerations

The Geotechnical Designer should assess the need for a geotechnical subsurface investigation. The Geotechnical Designer should collect previous subsurface data, field observations, performance data of the existing substructure, and typical soil characteristic tables to make a site-specific decision. In the event that enough information regarding the subsurface conditions exists, the Geotechnical Designer may choose to eliminate the subsurface investigation.

Even when the subsurface investigation is eliminated, design considerations (i.e., bearing capacity, settlement, frost protection, etc.) should be assessed by the Geotechnical Designer and made a part of the permanent record. When the subsurface investigation is eliminated from a project, it should be

understood that this will result in the need for a more conservative design and the use of higher factors of safety. The use of higher factors of safety may, in the end, be more economical than performing an in-depth subsurface investigation.

1.5.4 Hydrologic Considerations

Project characteristics that favor limited or no formal analysis of hydrology and hydraulics are found in Section 2.3.3 Level of Analysis.

1.5.5 Minimization of Approach Work

Limits of approach work, approach roadway width, guardrail upgrades, and surface treatments should be consistent with the adjacent roadway. Relaxation of design standards should be considered to achieve this consistency. The project length should be kept to an absolute minimum.

In considering relaxing these standards, the Designer should check with the Regional Transportation Advisory Committee (RTAC) representative in the Bureau of Planning to be certain that the corridor is not likely to be upgraded in the near future.

1.5.6 Reduction of Structural Design Effort

Structure type should be determined from Table 1-1 whenever feasible, instead of performing cost comparisons of various alternates in the Preliminary Design Report. Structures that do not meet the criteria would need to be custom designed.

A substructure should be designed to minimize stream impacts whenever possible, in view of typical short in-stream work windows. Consider using longer spans by placing the abutment behind an aging abutment that can adequately support the embankment, or choosing a replacement structure that does not require in-stream work. Minimize necessary work in the stream by founding the abutment above frost, if minor movement can be tolerable, or by choosing low impact structure types, such as pile bents or drilled shafts.

Table 1-1 Type/Material Selection Guide for Small Bridge Projects
Structure Type

Span Range	Bedrock at Site	Structure Type Determination	
10 to 21 ft	Bedrock	Plate Arch or Frame	Go to Materials Determination
	No Bedrock or Easily Removed	Pipe, Pipe Arch, or Box	Go to Materials Determination
22 to 26 ft	Bedrock	Frame	Go to Materials Determination
	No Bedrock or Easily Removed	Box	Go to Materials Determination
26 to 50 ft	NA	Concrete Arch, Concrete Frame, or Concrete Voided Slab	

Structure Material

Water or Soil Reactivity			Maintenance of Traffic During Construction	Material Determination
Salt or Brackish Water?	Soil or Water pH	If Existing Pipe is Steel, Age?		
Yes	5 to 9	NA	Close Road	Aluminum
Yes	5 to 9	NA	Staged	Concrete
Yes	<5 or >9	NA	NA	Concrete
No	6 to 8	< 40 years	Close Road	Aluminum
No	6 to 8	< 40 years	Staged	Concrete
No	6 to 8	> 40 years	Close Road	Galvanized Steel
No	6 to 8	> 40 years	Staged	Concrete
No	5 to 9	NA	Close Road	Aluminum
No	5 to 9	NA	Staged	Concrete
No	<5 or >9	NA	NA	Concrete

1.5.7 Contracting Strategies

The following strategies should be considered to reduce construction costs:

- o Grouping small projects for advertising can reduce costs. The projects should be located geographically near each other for efficiency of both MaineDOT personnel and the Contractor, and should be of similar scope. Projects from another Program sharing the same highway corridor or in the same general vicinity should also be constructed under one contract when feasible.

- o Simplify project details to allow for faster construction, especially for projects with short project schedules. Examples include the use of integral abutments, elimination of bridge skews, use of prefabricated superstructure elements, using uniform details, etc.
- o Time the bidding to allow enough time for the Contractor to plan their work. The advertisement of grouped projects should be far enough in advance of the construction season to allow as many Contractors to bid as possible.
- o Consider a reduced plan or no plan project. The project should have a well-defined scope, such as replacing an existing pipe with another pipe or pipe arch. There would be no survey obtained, and the plans would include: a typical pipe or pipe arch sheet, a typical roadway cross-section, and typical guardrail end treatments. These plans would be on standard letter size sheets that are inserted into the contract proposal book. For these projects, sufficient right-of-way must be available or easily attainable to construct the project, and minimal environmental impacts must be anticipated.

1.6 Non-Vehicular Bridges

A multi-use bridge may be constructed for a combination of pedestrians, bicyclists, snowmobiles, or other users. For loading criteria, refer to Section 3.8 Non-Vehicular Bridges. Prefabricated pedestrian bridges must be designed by a registered Professional Engineer.

The owner and maintainer of the bridge should consider the following issues when developing the design:

- o Width - For guidance on how wide a trail bridge should be, refer to AASHTO "Guide for the Development of Bicycle Facilities." A width less than 10 feet will prevent most vehicles from getting on to the bridge except for snowmobiles, ATV's, golf carts, and motorcycles. If the bridge will be plowed, additional width may be necessary.
- o Vertical clearance - Vertical clearance is an issue with timber covered bridges or box type steel trusses. The minimum vertical clearance is 8 feet. Low vertical clearance will prevent heavier vehicles from using a bridge. A high vertical clearance of 14'-6" or more may be needed to accommodate snow grooming equipment, occasional maintenance equipment, or emergency vehicles.
- o Emergency Vehicles – If emergency vehicles (ambulances, fire trucks, etc.) are expected, they should be accommodated. The bridge may be the only access to a remote area.

- Inspection/Maintenance - How will the bridge be inspected and repaired? Refer to Section 2.9.6 Maintainability.
- Bollards – Bollards may be used to control or limit access. Bollards are usually timber or steel posts spaced at about 5 foot spacing that prevent large vehicles from going onto a bridge. The spacing of the bollards can be reduced to 3 feet clear to prevent virtually all motorized vehicles from using the bridge. Removable bollards should be considered if emergency or maintenance vehicles will occasionally use the bridge.
- Rail - Bridges that may be used by snowmobiles should use at least a 42" bicycle height bridge rail. The use of a rub rail is highly recommended to prevent bicycle handlebars from catching on the bridge rail.

The Structural Designer should also consider the use of security fencing, lighting, and attached utilities on the bridge. The load capacity of the bridge should be clearly posted on or near the bridge in accordance with MUTCD.

1.7 Aesthetics

1.7.1 General

Aesthetics involves more than just surface features such as color and texture. It includes the visual and perceptual effect made by the bridge as a total structure, as well as the effect made by its individual parts. Bridges affect their surroundings by virtue of their size, shape, line, color, and texture. All structures should be designed with consideration of site-specific features to create designs that provide function as well as a pleasing appearance. The key is to create a distinguished structure without spending excessive resources.

Bridges are usually viewed from one of two places, either from the roadway as a user, or from the side. For those bridges rarely seen from the side, aesthetic considerations are limited to the appearance of the rail, sidewalk, curb, and wearing surface. For other bridges, the view of the bridge from the side should be considered in the design. The nature of the surroundings may influence the aesthetic design choices, whether the location is urban, rural, industrial, or coastal.

1.7.2 Design Considerations

Consistency in the use of flares and tapers in bridge components will result in a more harmonic structure. For example, if a column is flared to be wider at

the top, the fascia should also be sloped. A prismatic column may look better with a vertical fascia. Refer to Figure 1-1.

1.7.2.1 Superstructure

A bridge is primarily a horizontal structure that is supported by vertical members. Fewer piers will enhance the appearance by emphasizing the horizontal line. End spans that are shorter than middle spans often have structural as well as aesthetic advantages. A constant depth superstructure will appear more graceful than one with spans of different depths. Even more graceful is a haunched girder structure, especially if the haunch transition is long, up to 40% of the span length.

The end of the slab seen on the fascia will look better if it appears thinner. This can happen by creating deeper shadows through sloping the bottom of the fascia away from the viewer, or tapering the slab thickness toward the fascia, and by using an overhang of about 2/3 the depth of the girder. Refer to Figure 1-2 and Figure 1-3.

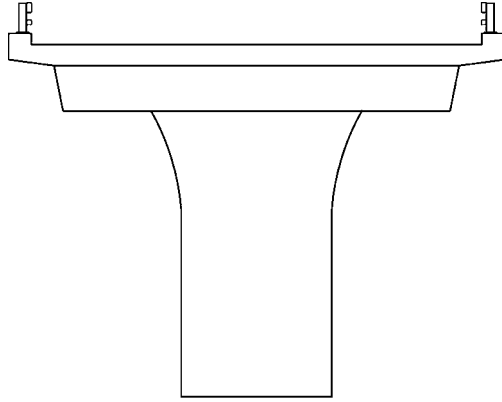
The rail may be the most visible aspect of the bridge to the traveling public. Spending money enhancing the rail system can go far to improve the appearance of the structure. Refer to Section 4.4.6 Aesthetics.

Ornamental lighting can enhance the aesthetics of a high profile bridge. Tall light poles can be located over piers to streamline the appearance.

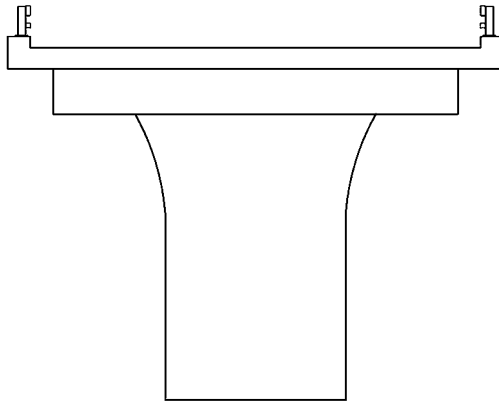
1.7.2.2 Substructure

Most piers are classified as short, with the length (transverse) greater than the visible height. It is more difficult to enhance the appearance of a short pier than a tall pier. The vertical nature of a tall pier can be emphasized by minimizing the batter, and by minimizing the horizontal faces of the pier by using sloped faces. When a bridge has several piers with different heights, the pier shape should be one that can accommodate varying proportions and batters to create both short and tall piers that look good. Batters can be greater on a short pier without sacrificing appearance.

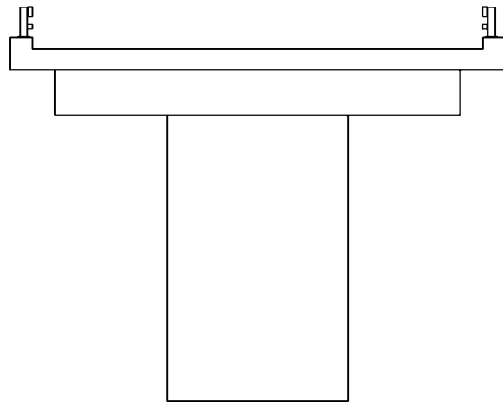
Hammerhead piers should be proportioned to balance the shaft length and height, as well as the length and depth of the cantilevered cap. Some starting proportions are shown in Figure 1-4. The Structural Designer should do a scale drawing of each pier to be sure the proportions look pleasing. A short cantilever looks better when the shaft batter is negative toward the ground, while a longer cantilever is needed when the batter is positive. Refer to Figure 1-5.



Acceptable

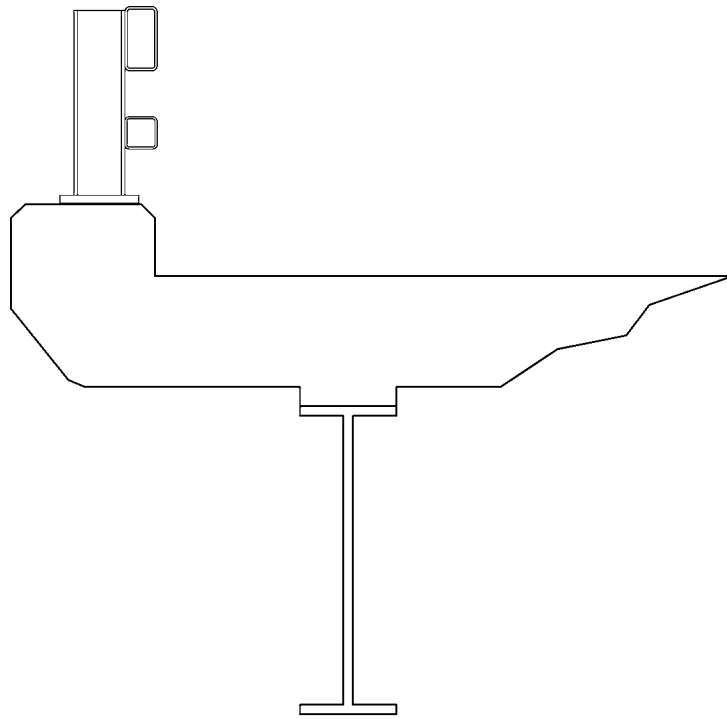


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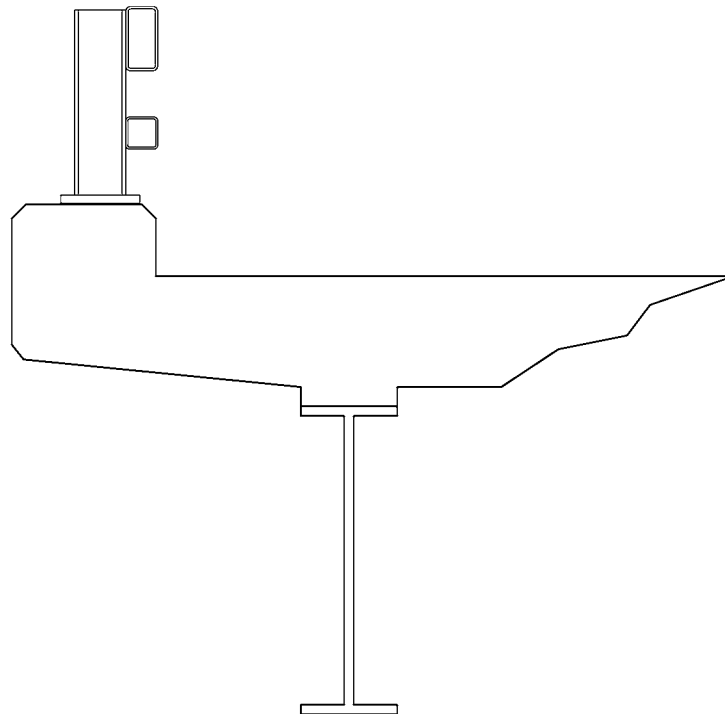


Acceptable

Figure 1-1 Consistent Use of Flares



Sloped Fascia



Tapered Slab

Figure 1-2 Methods to Thin Appearance of Fascia

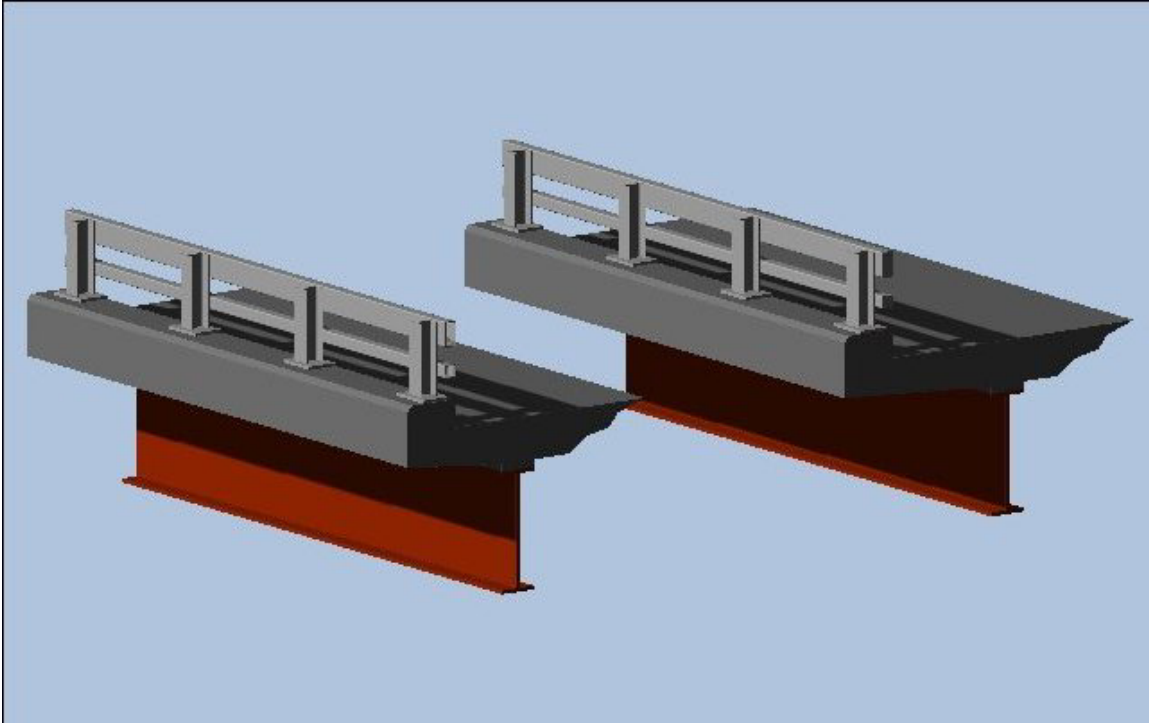


Figure 1-3 Effect of Overhang Length on Beam Shadow

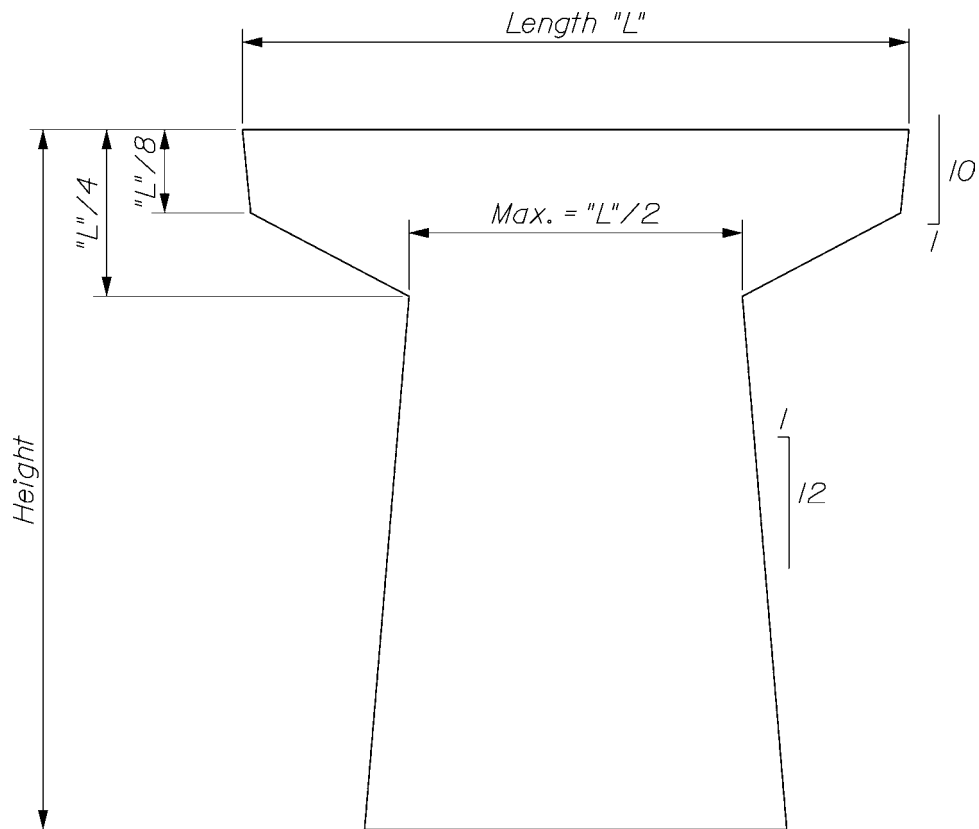


Figure 1-4 Hammerhead Pier Proportions

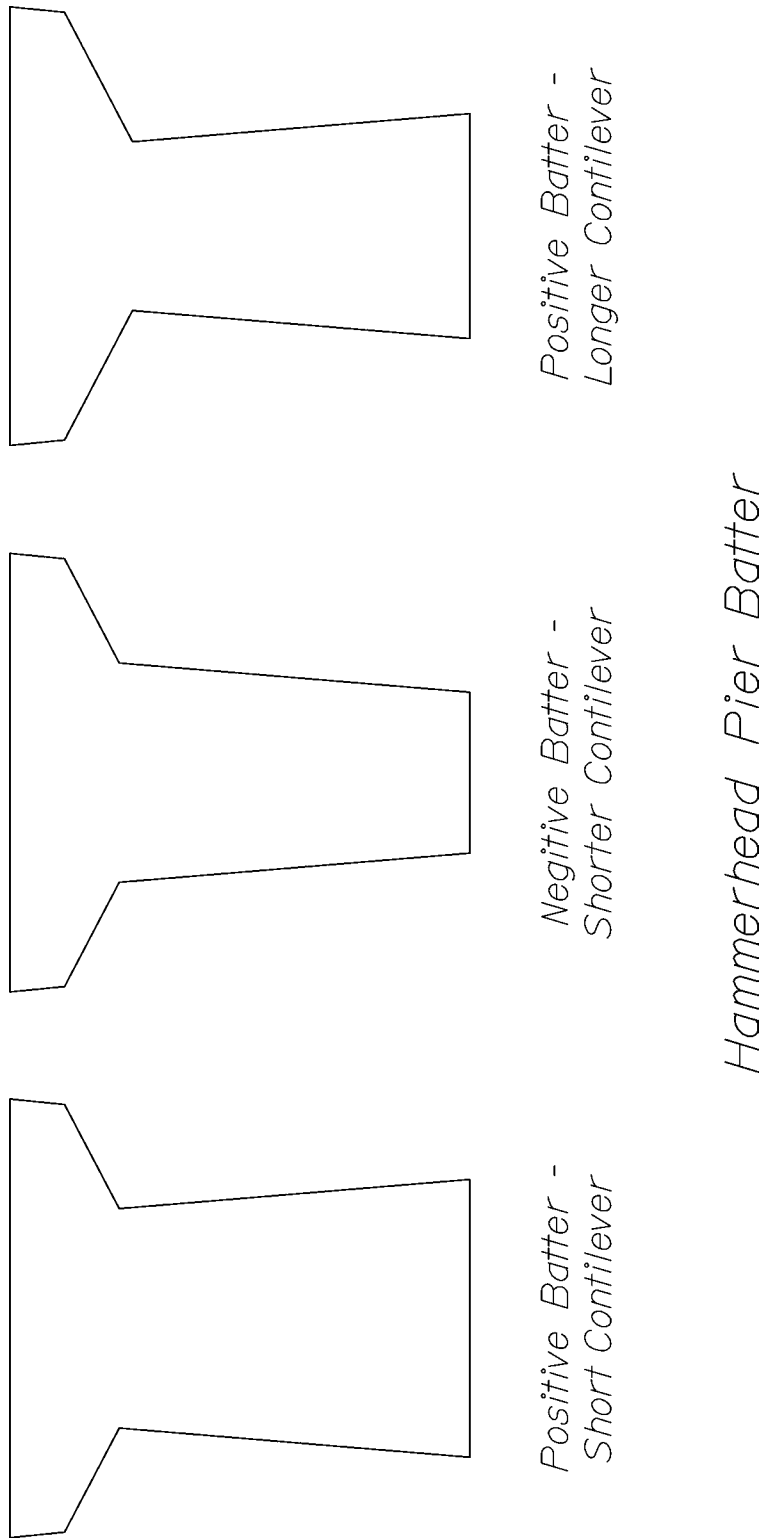


Figure 1-5 Variations of Cantilever Length and Batter

The relative pier width (longitudinal) to the fascia depth seen from the side also affects appearance. If the pier is too narrow, the bridge will appear unsupported and weak, while a wide pier will appear bulky. The apparent fascia depth includes the parapet rail height for a closed rail system, but does not include the rail height for an open rail system. Pier width should be between 25% and 50% of the fascia depth for a concrete barrier system. It should be between 50% and 67% of the fascia depth for an open rail system. Refer to Figure 1-6.

In general, slender columns are more graceful than wider columns. Columns will look more slender if the edge facing the viewer is partially in shadow. An octagonal column may look thinner than a round column, which looks thinner than a square column. Refer to Figure 1-7.

Form liners, acid washing techniques, or stone facing can be used to create surface texture on abutments, wingwalls, and piers. If the wall is viewed only at high speeds, the patterns used must be large enough to be visible. Pay special attention to corners and tops of walls when imitating stonework with form liners. Also consider having horizontal lines on return wings such as chamfers and construction joints follow the road grade when possible.

1.7.2.3 Color

In special situations, adding color to components of the bridge can be considered to enhance the fit into the surroundings. Coloring will increase maintenance costs, and may result in a poor appearance if maintenance is neglected. Concrete can be colored, but the cost is high, quality control is difficult, and it is often hard to match colors between batches. Concrete can also be stained, which presents its own appearance and durability concerns. Steel bridge rail can be color galvanized, as discussed in Section 4.4.6 Aesthetics, Bridge Rail, and other steel structures such as historic trusses can be painted as well.

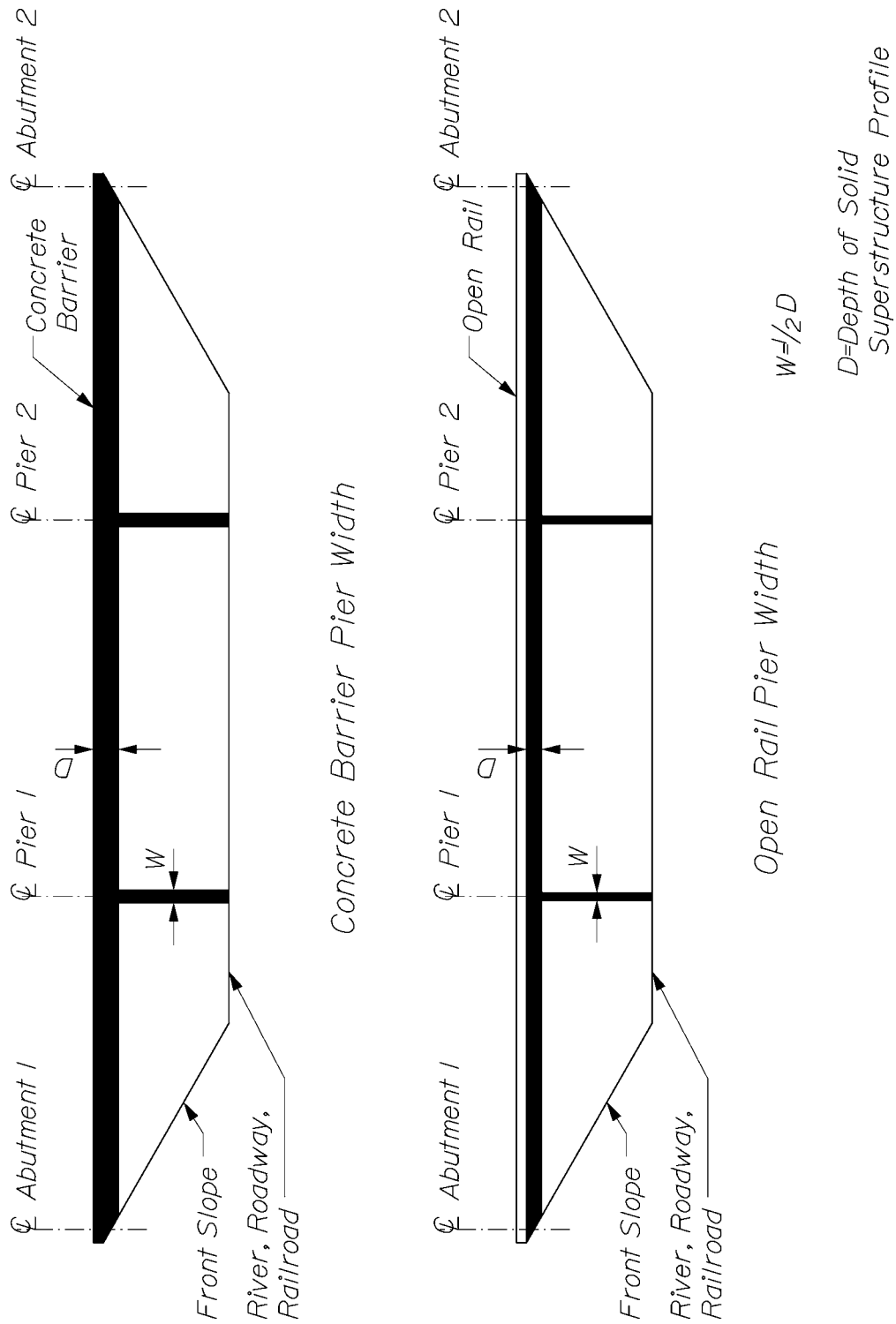


Figure 1-6 Ratio of Pier Width to Fascia Depth

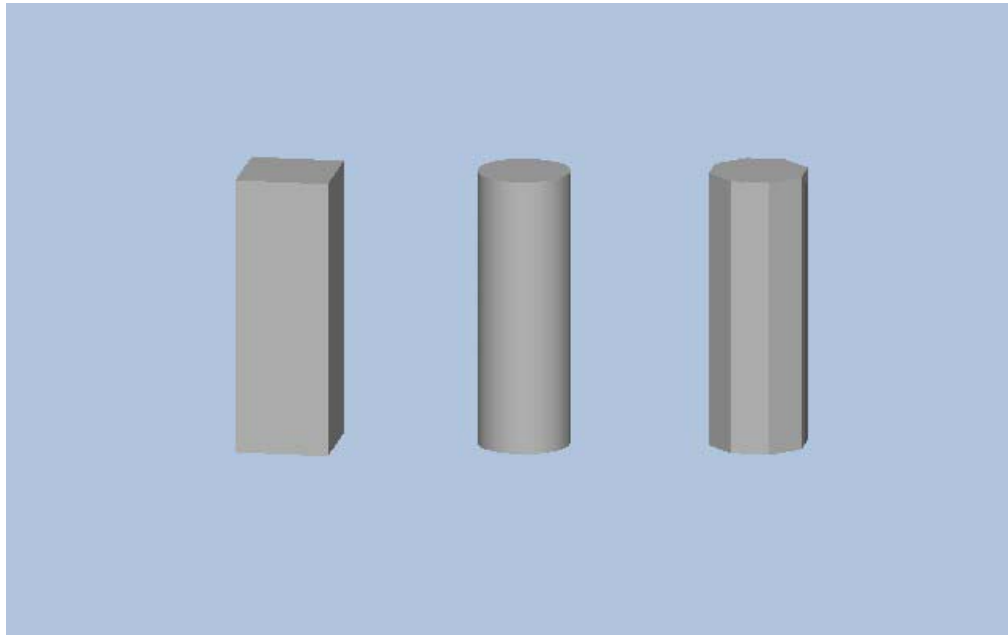


Figure 1-7 Effect of Column Shape on Shadows and Thin Appearance

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Chapter 2

PRELIMINARY DESIGN



Chisholm Park Bridge, Rumford



Sandy Stream Bridge, Moose River Plantation

2	PRELIMINARY DESIGN.....	2-1
2.1	Preliminary Design Report.....	2-1
2.1.1	Title Page	2-2
2.1.2	Table of Contents	2-2
2.1.3	Background Information	2-2
2.1.4	Existing Bridge Synopsis Form.....	2-2
2.1.5	Location Map.....	2-3
2.1.6	Bridge Recommendation Form.....	2-3
2.1.7	Summary of Expected Impacts.....	2-6
2.1.8	Summary of Preliminary Design	2-6
2.1.9	Hydrology/Hydraulic/Scour Report	2-7
2.1.10	Preliminary Plan	2-8
2.1.11	Photographs	2-8
2.1.12	Summary of Existing Upstream and Downstream Bridges	2-9
2.1.13	Site Inspection Report	2-9
2.1.14	Information Reports	2-9
2.1.15	Survey Plans of Existing Bridges.....	2-9
2.1.16	Hydrology/Hydraulic/Scour Data	2-9
2.1.17	Miscellaneous Information.....	2-9
2.1.18	Traffic and Accident Data	2-10
2.1.19	Estimates.....	2-10
2.2	Economic Comparisons.....	2-10
2.2.1	Overview	2-10
2.2.2	Definition of LCCA.....	2-11
2.2.3	When to use LCCA.....	2-11
2.2.4	Deterministic Analysis	2-12
2.2.5	Probabilistic Analysis.....	2-12
2.2.6	Standard Assumptions	2-12
2.2.7	Cost Comparison for Number of Beams.....	2-14
2.3	Hydrology, Hydraulics, and Scour	2-16
2.3.1	General.....	2-16
2.3.2	Minor Span/Strut Determination	2-16
2.3.3	Level of Analysis.....	2-17
2.3.3.1	Level 1 (Qualitative Analysis).....	2-17
2.3.3.2	Level 2 (Basic Analysis).....	2-18
2.3.3.3	Level 3 (Complex Analysis)	2-18
2.3.4	Data/Information Collection	2-18
2.3.5	Vertical Datum.....	2-21
2.3.6	Tidal Elevation Computations.....	2-22
2.3.7	Changes in Sea Level	2-26
2.3.8	Documentation	2-26
2.3.9	Hydrology	2-26
2.3.9.1	Introduction	2-26
2.3.9.2	Discharge Rate Policy.....	2-27
2.3.9.3	Discharge Formulae.....	2-27
2.3.9.4	Rural Watersheds	2-28

CHAPTER 2 – PRELIMINARY DESIGN

2.3.10	Hydraulics.....	2-29
2.3.10.1	Introduction	2-29
2.3.10.2	Hydraulic Analysis	2-29
2.3.10.3	Discharge Velocities	2-33
2.3.10.4	Backwater	2-33
2.3.10.5	Dams.....	2-33
2.3.10.6	Fish Passage.....	2-34
2.3.11	Scour	2-34
2.3.11.1	New Bridges	2-34
2.3.11.2	Existing Bridges.....	2-35
2.3.11.3	Riprap Slope Protection	2-36
2.4	Maintenance of Traffic During Construction.....	2-37
2.4.1	General.....	2-37
2.4.2	Methods to Maintain Traffic	2-38
2.4.2.1	Close the Road and Detour on Existing Roads	2-38
2.4.2.2	Staged Construction.....	2-39
2.4.2.3	Temporary Bridge.....	2-39
2.4.2.4	Innovative Methods	2-39
2.5	Geotechnical and Survey.....	2-40
2.5.1	Geotechnical	2-40
2.5.2	Field Survey	2-40
2.6	Utilities and Right-of-Way	2-42
2.7	Alignments.....	2-42
2.7.1	General Highway Design Guidelines	2-42
2.7.2	Bridge Guidelines	2-43
2.7.2.1	Horizontal Alignment	2-43
2.7.2.2	Vertical Alignment	2-43
2.7.3	Clearances	2-43
2.7.3.1	Railroad.....	2-43
2.7.3.2	Grade Separations	2-45
2.7.3.3	Underclearance for Stream Crossings	2-45
2.7.3.4	Clearance Between Parallel Structures.....	2-45
2.7.3.5	Underclearances for Non-Vehicular Bridges	2-46
2.8	Approaches	2-46
2.8.1	Roadway Widths.....	2-46
2.8.1.1	Local Roads	2-46
2.8.1.2	Collector Roads.....	2-48
2.8.1.3	Arterials	2-48
2.8.2	Guardrail.....	2-48
2.8.2.1	General	2-48
2.8.2.2	Guardrail Treatment on Local Roads	2-48
2.8.3	Reduced Berm Offset.....	2-56
2.8.4	Pavement Design	2-57
2.8.4.1	General	2-57
2.8.4.2	Arterials and Collectors	2-58
2.8.4.3	Local Roads	2-59

CHAPTER 2 – PRELIMINARY DESIGN

2.8.5	Approach Drainage.....	2-60
2.8.6	General or Local Conditions	2-60
2.9	Structure Type Selection	2-61
2.9.1	Span	2-61
2.9.2	Maintenance of Traffic.....	2-61
2.9.3	Constructability	2-61
2.9.4	Environmental Impact.....	2-62
2.9.5	Right-of-Way Impact.....	2-62
2.9.6	Maintainability.....	2-63
2.9.7	Historical/Archeological Issues.....	2-64
2.9.8	Cost.....	2-64
2.9.9	Aesthetics.....	2-64
	References	2-65
	Table 2-1 Rounding Guidelines for PDR Cost Estimates.....	2-10
	Table 2-2 Life Cycle Intervals	2-13
	Table 2-3 Design Flow versus Drainage Area and Wetland Percent	2-17
	Table 2-4 Clear Zone	2-52
	Table 2-5 Encroachment Angle.....	2-52
	Table 2-6 Pavement Layer Thickness.....	2-57
	Table 2-7 Number of Layers Across Roadway	2-58
	Table 2-8 DARWin Input.....	2-59
	Table 2-9 Pavement & Subbase Thickness	2-60
	Figure 2-1 Typical Railroad Cut Section	2-44
	Figure 2-2 NHS in Maine	2-47
	Figure 2-3 Point of Need Definition.....	2-50
	Figure 2-4 Lateral Extent of Hazard Definition	2-51
	Figure 2-5 Point of Need Example.....	2-55
	Figure 2-6 Lateral Extent of Hazard Example	2-56
	Figure 2-7 Reduced Berm Offset.....	2-57
	Procedure 2-1 Guardrail End Treatment on Local Roads	2-51
	Example 2-1 Cost Comparison of Number of Steel Beams	2-15
	Example 2-2 Datum Shift.....	2-22
	Example 2-3 Tidal Elevation at Reference Station.....	2-23
	Example 2-4 Tidal Elevation at Subordinate Station	2-24
	Example 2-5 Guardrail End Treatment on Local Roads.....	2-54

2 PRELIMINARY DESIGN

2.1 Preliminary Design Report

The Preliminary Design Report (PDR) documents the justification for decisions made in the conceptual design process. Forms are available electronically that assist in completing the PDR. At the end of the preliminary design phase, all those invested in the project have reviewed the scope of work, and this scope is considered final. The PDR is then used as the starting point to proceed to final design.

For those projects with spans of 50 feet or less, consideration should be given to a reduced preliminary design effort, as discussed in Section 1.5 Small Bridge Initiative.

The PDR is organized into the following sections. The depth of study and extent of investigation of options will depend upon the complexity of the project. A description of each section follows the listed sections.

1. Title Page
2. Table of Contents
3. Background Information
4. Existing Bridge Synopsis Form
5. Location Map
6. Bridge Recommendation Form
7. Summary of Expected Impacts
8. Summary of Preliminary Design
9. Hydrology/Hydraulic/Scour Report
10. Preliminary Plan
11. Photographs
12. Summary of Existing Upstream and Downstream Bridges
13. Site Inspection Report
14. Information Reports
15. Survey Plans of Existing Bridges
16. Hydrology/Hydraulic/Scour Data
17. Miscellaneous Information
18. Traffic and Accident Data
19. Estimates

For routine maintenance-type projects such as bridge wearing surface replacements and bridge painting, a one-page “short form” PDR may be used in lieu of the standard forms and sections listed. When warranted, additional information about the project can be attached to this form. Electronic Microsoft Word templates of some common PDR layouts are available on the Department’s website.

2.1.1 *Title Page*

The Title Page contains the following:

Preliminary Design Report
Bridge Name and Number
over
River Name
Town, Maine
Federal Project Number
WIN Number

2.1.2 *Table of Contents*

This should be a properly identified index of pages both for sections of the PDR forms and for appendices.

2.1.3 *Background Information*

This page provides a quick reference for background information on the project. Much of this information is found either in MaineDOT's ProjEx, the Planning Report, or Bridge Management's SI&A sheet, all of which will be provided by the Project Team. The following sections are completed as shown below:

Program Scope: Copy verbatim the scope from the Work Plan.

Program Description: Copy verbatim the contents of the project description in the Work Plan.

Project Background: Provide a brief written description of the project's background covering any previous studies and recommendations, requests by Towns, and any other pertinent information.

2.1.4 *Existing Bridge Synopsis Form*

This form provides a description of the physical characteristics, history, and condition of the existing structure and should be filled in as completely as possible from information in Bridge Maintenance files and project records. Some terms are defined as follows:

Structurally Deficient: A structure is structurally deficient if the condition rating for the deck, superstructure, substructure, or the culvert and retaining wall is 4 or less. A structure may also be structurally deficient if

the appraisal rating for the overall structural condition or waterway is 2 or less.

Functionally Obsolete: A structure is functionally obsolete if the appraisal rating for the deck geometry, under clearances, or approach roadway alignment is 3 or less. A structure may also be functionally obsolete if the appraisal rating for the overall structural condition or waterway is 3. Any bridge classified as structurally deficient is excluded from the functionally obsolete category.

2.1.5 Location Map

This should be from the Maine DOT Map Viewer or another reliable source of road and terrain maps. Do not use copyrighted material such as a DeLorme's Maine Atlas and Gazetteer. Use of images from Google Maps is acceptable under their Terms of Service in November 2015, but that could change at any time that Google wishes.

2.1.6 Bridge Recommendation Form

All portions of the Recommendation Form should be completed as shown below. A complete description of each component should be included under that component. There are several variations to this form depending on the project scope. If there are parts that are not applicable to the structure type, they need not be included.

Approved by – The Senior Structural Designer and the Assistant Bridge Program Manager for Design must both sign off on the PDR

Project - State the type of project. Examples:

“Bridge replacement with 300 ft of approaches, including transitions”

“Bridge rehabilitation project with no approach work”

“Bridge replacement as part of Highway Program project”

“Bridge replacement with approaches by Highway Program”

Alignment Description - Give a description of the horizontal and vertical alignments at the structure location and the relationship to the existing alignment. Example:

"1200' horizontal curve located approximately 30' upstream of existing bridge and a 500' sag (crest) vertical curve with a finish grade 3.5' higher than existing bridge."

Approach Section - Give a description of the typical approach section at the bridge, including the type of guardrail. Example:

“Two 11' paved lanes with 3' shoulders (30' rail-to-rail) with standard sideslopes. 21" aggregate subbase course gravel with 3" pavement thickness. Type 3 guardrail.”

Spans - Give the span lengths along the centerline of construction on straight tangents, and along working lines or chord lines for structures on a curve. If on a curve, indicate span lengths as "along long chord" or other descriptive indication. This section is not required for culvert-type structures.

Skew - Give the skew angle of the substructure units, or the centerline of a culvert-type structure, relative to the longitudinal working line of the structure. The skew angle should always be given as "Ahead on Left" or "Back on Left".

Loading - Indicate the appropriate design vehicle loading.

For a typical superstructure:
“HL – 93 Modified for Strength 1”

Superstructure - Give the design description and governing parameters of the superstructure. For culvert-type structures, this section is simply called Structure. Examples:

For a typical superstructure:
“Five rolled beams of A709, Grade 50W steel with a composite structural concrete slab, elastomeric bearings, one compression seal expansion joint, and a 3" bituminous wearing surface with ¼" (nominal) membrane waterproofing. 36' curb-to-curb with standard 2-bar steel rail. 2% normal crown.”

For a culvert-type structure:
“16'-4" span by 8'-2" rise aluminum structural plate pipe arch. Flow line of 1% with Elevation 100.00 at the centerline of construction.”

Abutments - State the type of abutment and anticipated support system. Also give any specific features required. This section is not required for culvert-type structures. Example:

"Stub concrete abutments with return wings on steel H-piles, 1.75:1 (plain or heavy) riprap slopes in front" or "Deep concrete abutments

with approach slabs on spread footings with sandblasted architectural facing".

Piers - State the type of piers and anticipated support system. This section is not required for culvert-type structures. Example:

"Mass concrete pier with distribution slab and concrete seal supported by steel H-beam piles."

Opening and Clearance - For water crossings, give the total area of bridge opening and the area of bridge opening at a common elevation for both the existing and the recommended structures. The areas should be normal to the direction of flow. Also, give the minimum clearance depth at Q50 for both the existing and the recommended structures.

For overpass structures, give the minimum vertical and horizontal clearances for both the existing and the recommended structures.

For culvert-type structures, give the total opening for both the existing and the recommended structures.

Available Soils Information - State what soils information was available during study or was obtained from existing plans. Also indicate if scour analysis should be made in the final design of the foundation.

Additional Design Features - Describe any design features that are not described in any other part of the Recommendation Form (e.g. something that is unusual or experimental), but which are necessary to complete the project description.

Maintenance of Traffic - State how and where traffic is to be maintained during construction of the project, whether one lane or two lanes will be required, and whether signals or flaggers will be required. Also state if maintenance of pedestrian traffic is required. If a road closure is proposed, give the detour length from abutment to abutment.

Construction Schedule - Include any restrictions and/or commitments. Example:

"One construction season with landscaping the following spring. Bridge must be reopened to traffic by Labor Day."

Advertising Date – The current estimated advertise date available in Projex.

Estimated Project Cost - Enter the programmed, available, and the estimated project costs under the appropriate headings.

Commentary: The estimated cost of the project is located in 3 places within the PDR: here in the program funding table, summary of preliminary design, and the cost estimate.

Additional Borings Required and Additional Geotechnical Evaluations Required - Indicate whether or not the information is required.

Approved Design Exceptions - List any approved or pending exceptions to Federal or State Standards that either requires approval from FHWA (for NHS projects only), the Engineer of Design, or the Engineering Council. Examples of exceptions to standards are reduced bridge widths, omitting of the leveling slab on butted precast superstructures, and reduced hydraulic clearances.

2.1.7 *Summary of Expected Impacts*

This form provides a summary of the expected impacts and the required permitting for the recommended project. These impacts may be right-of-way, utilities, historical, archeological, environmental, etc. The required permitting may include Coast Guard, FAA, and the various environmental permits. Filling in the required information for this form will be a project team effort.

2.1.8 *Summary of Preliminary Design*

This is a summary of the Preliminary Design performed to determine the project recommendations. It should describe, in an orderly fashion, the alternatives considered, with a summary of the assumptions and comparisons that are pertinent to the justification of the recommendation. It should include a discussion of bridge width, alignment, and maintenance of traffic, with the reasoning used to arrive at the recommendation. It may include a discussion of geotechnical, environmental, or utility issues, if these are pertinent to the project.

The Summary should discuss the pros and cons of the alternatives considered and the reasons for the selection of the recommended alternative. Only the engineering that is pertinent should be discussed. The Summary should be short and to the point and should avoid superfluous and lengthy discussions.

For a water-crossing structure, reference should be made to the Hydrology/Hydraulic/Scour Report with the conclusions repeated as to the feasible structure alternatives and ultimate recommendation.

In some instances, especially on large and expensive projects, there may be several alternatives developed for public or internal review and selection.

These alternatives should be summarized here, with the back-up data and calculations bound and filed elsewhere in the project file.

2.1.9 Hydrology/Hydraulic/Scour Report

This is a summary of the hydrologic analysis that determines the design and check discharges, the hydraulic analysis that determines the structure opening and/or structure alternatives, and the scour analysis that determines the foundation requirements. Normally, this report combines the Hydrology and Hydraulics, but it can be separated into two reports if warranted. The MaineDOT Environmental Office Hydrology Unit provides a spreadsheet with the results of the U.S.G.S. full regression equation. Flows based on other methods should be computed and documented by the Designer. These flows are summarized in this section. Example:

Drainage Area	110 sq mi
Design Discharge (Q50)	1240 cfs
Check Discharge (Q100)	1410 cfs
Scour Check Discharge (Q500)	1660 cfs
Ordinary High Water (Q1.1)	380 cfs
Flood of Record (Q---)	1820 cfs @ Elevation 64.3

If HEC-RAS runs will be necessary for the hydraulic study, stream slopes should be determined. If the structure is in a tidal zone, the following elevation data should also be summarized:

Mean Lower Low Water (MLLW)	-8.5 ft
Mean Low Water (MLW)	-8.2 ft
Mean Tide Level (MTL)	-0.3 ft
Mean High Water (MHW)	7.5 ft
Mean Higher Water (MHHW)	9.4 ft
2003 Predicted High Tide	10.7 ft

The hydraulic analysis is then discussed. Structural openings should be analyzed for flow capacity, outlet velocities, and backwater heights, using Bureau of Public Roads (BPR) charts and graphs, backwater runs, or other applicable methods. Culvert-type structures should be checked for fish passage at low flow conditions.

If no single structure alternative is obvious, the Hydrology/Hydraulic/Scour Report should describe those alternatives that are hydraulically feasible, and the final recommended alternative should be discussed in the Summary of Preliminary Design of the Bridge Recommendation Form.

A summary gives the final conclusions and hydraulic parameters. Also, for comparative purposes, the Summary should give the hydraulic parameters of the existing bridge. Example:

	Existing Bridge 60 ft clear span	Recommended 88 ft clear span
Headwater El. @ Q50	104 ft	101 ft
Headwater El. @ Q100	107 ft	102 ft
Discharge Velocity @ Q50	9.1 fps	5.2 fps
Discharge Velocity @ Q100	12.6 fps	6.5 fps
Ordinary High Water (Q1.1)	98.1 ft	98.1 ft
Discharge Velocity @ Q1.1	3.5 fps	2.0 fps
Clearance @ Q50	1.3 ft	4.2 ft

2.1.10 Preliminary Plan

A half-size copy of the Preliminary Plan will be added to the PDR after its preparation and it should be included in the Table of Contents. Typical sections of existing and proposed bridges should be shown on the Preliminary Plan, as well as proposed construction and other pertinent data.

2.1.11 Photographs

A good selection of color photographs of the bridge, roadway, and stream should be taken during a field inspection visit or from photographs taken by others. Photographs may also be copied from the Bridge Maintenance files or

obtained from local residents taken during a flood or during the construction of the existing bridge. When possible, the date the photographs were taken should be noted.

2.1.12 Summary of Existing Upstream and Downstream Bridges

Information about the upstream and downstream bridges may be useful for the hydraulic analysis. If so, they are listed here along with the size of the hydraulic opening and pertinent ice, flooding, and debris concerns.

2.1.13 Site Inspection Report

All field trips to the project site should be documented, describing all pertinent findings, conclusions, and points of interest.

2.1.14 Information Reports

Reports from Bridge Maintenance Supervisors, local residents, or Town Officials pertaining to structural condition or hydraulics should be documented. A copy of the most recent inspection report should also be included here.

2.1.15 Survey Plans of Existing Bridges

Archived survey or general plans of the existing bridge should be printed and included here. Plans of nearby bridges may also be included if they have pertinent information related to flood history, soils, or topography which could be used in the preliminary design. Pertinent structural plans may also be included for complex rehabilitation projects when deemed beneficial.

2.1.16 Hydrology/Hydraulic/Scour Data

This section provides the back-up data to the Hydrology/Hydraulic/Scour Report, such as the flow data tabulation, aerial photographs, analysis of existing bridges, FEMA data, BPR hydraulic graphs and charts, HY-8 results, HEC-RAS results, scour computations, and other relevant information. If the project has extensive computer reports from the hydraulic analysis, include the most pertinent information in the PDR. Additional hydrology/hydraulic/scour data should be compiled in a separate document, placed in the project file, and referenced in the PDR.

2.1.17 Miscellaneous Information

Any other pertinent information that is developed or obtained can be included here.

2.1.18 Traffic and Accident Data

The traffic data information obtained from the Bureau of Planning is included here. Include accident data if pertinent to the project.

2.1.19 Estimates

Preliminary Cost Estimate forms are available electronically to assist in estimate preparations. They should be included here for all developed alternates. Supporting spreadsheets that estimate costs using detailed pay items should not be included in the PDR; however, they can be placed in the project file. As a check on the accuracy of the estimate, the square foot cost obtained should be compared to historical square foot cost data found in the Bridge Program’s Bridge Unit Cost database. All project costs should be rounded as shown in Table 2-1.

Table 2-1 Rounding Guidelines for PDR Cost Estimates

Item	Amount	Round To Nearest:
Individual construction items such as Superstructure, Cofferdams, Approaches, Mobilization, etc.	All	\$1,000
Structure Subtotal and Approaches Subtotal	All	\$5,000
Total Construction Cost, PE, ROW, CE	Up to \$1,000,000	\$5,000
	Over \$1,000,000	\$10,000
Total Project Cost	Up to \$500,000	\$5,000
	\$500,000 to \$1,000,000	\$10,000
	Over \$1,000,000	\$100,000

2.2 Economic Comparisons

2.2.1 Overview

During preliminary design, the Designer should consider different rehabilitation/replacement alternatives. A Life Cycle Cost Analysis (LCCA) is a tool used to select alternatives and to make economic decisions. Sound

engineering judgment is necessary to determine input data, analyze results, and determine the relevance of the analysis.

LCCA considerations for bridges include functionality, age, condition, present costs, future costs, and present and future program funding availability. The two approaches available to evaluate LCCA are a Deterministic Analysis and Probabilistic Analysis. This section will examine both analyses.

2.2.2 Definition of LCCA

Section 303 of the National Highway System Designation Act defines LCCA as “a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future cost, such as maintenance, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment”.

In short, LCCA is a method of analysis that compares the net present value of all costs related to improvements over the life of the structure. The level of detail of the analysis is determined on a project-by-project basis.

2.2.3 When to use LCCA

LCCA should be performed when comparing competing options with different life expectancies, rehabilitation costs, or maintenance costs. Common situations are listed below:

- o A rehabilitation scenario for a single bridge with multiple choices such as: 1) immediate deck replacement; 2) wearing surface replacement followed in 15 years by a deck replacement; 3) deck rehabilitation and wearing surface replacement followed by a superstructure replacement in 15 years; etc. (refer to Chapter 10 Rehabilitation for a discussion of this terminology)
- o Comparing a traditional bridge that has significant maintenance costs to a buried structure that has few maintenance costs
- o Bridge rehabilitation compared with replacement
- o Painting a bridge or waiting until the bridge is deficient and then replacing it
- o Comparing steel bridge that requires painting with a concrete structure that is to be located in a harsh environment where weathering steel is not recommended
- o Comparing a steel pipe to an aluminum pipe or concrete box

2.2.4 Deterministic Analysis

A deterministic analysis is the most common method, and is adequate to evaluate LCCA in most situations. This approach compares alternatives and life cycle costs based on net present value and fixed inputs. This simplified approach will provide one solution for any given set of alternatives. To vary costs or timing, inputs need to be changed and the analysis rerun. For most projects the inputs can be easily adjusted utilizing a spreadsheet. Design examples are available in Excel from the technical resource people for economic comparisons.

2.2.5 Probabilistic Analysis

The next level of LCCA is a probabilistic analysis. This approach allows for variability and uncertainty of timing and costs. The output provides a probability of which alternate will have the lowest costs over the life of the bridge. This method of analysis is recommended for projects with significant bridge replacement or rehabilitation costs, or when the deterministic approach is insufficient.

The Bridge Program utilizes a program developed by NCHRP Project 12-43. Bridge Life Cycle Cost Analysis (BLCCA) has the ability to perform both a probabilistic and a deterministic analysis. BLCCA can be installed on the Designer's PC as needed. A complete Guidance Manual and User's Manual is also available for reference that can be viewed and printed through the help menus.

2.2.6 Standard Assumptions

To ensure consistency the following assumptions are recommended:

- o Use a discount rate of 4%, which approximates the FHWA discount rate. This factor accounts for the annual growth rate of an investment, and does not include inflation.
- o Use current and constant dollars. For example, if the cost for a repair in year 1 is \$100,000, the same repair in year 10 will also cost \$100,000.
- o Routine maintenance costs are assumed to be the same for all alternates and are ignored in the analysis, except when comparing different structure types such as a buried structure to a traditional bridge. These costs include such activities as minor wearing surface and concrete repairs, yearly cleaning of bearings and drains, and repair of damaged railings.

- o User costs are assumed to be the same for all alternates and are ignored in the analysis, unless one alternate has a significant impact on the public over another alternate. User costs can be requested from Planning, if they are used in the analysis.
- o Suggested rehabilitation intervals over the life of the bridge are shown in Table 2-2. These may be used as a guide in developing the future rehabilitation over the life of an existing or proposed bridge.
- o The Designer should not rely solely on LCCA. The results from LCCA always show deferring costs as the most cost effective solution. However, it is important to consider the additional costs to maintain an old bridge, the impact to the traveling public as a result of additional maintenance work, risks associated with a deteriorating structure, and availability of funding when replacement becomes absolutely necessary. The functionality of the bridge is also important. Replacing a bridge to modern standards may provide an increased bridge width, new sidewalks, or an improved alignment.

Table 2-2 Life Cycle Intervals

Capital Investment	Useful Life of Component (years)
Wearing Surface Replace/Rehab	15
Deck Rehabilitation (includes wearing surface)	30
Deck Replacement	50
Bridge Replacement	75
Painting	Refer to Section 7.2.3 Coatings
Sliplining	Depends on materials used and site conditions
Invert Lining	25+
Steel Pipe	50
Plastic Pipe	100
Aluminum Pipe	75
Concrete Pipe/Box	75-100

Notes:

1. Condition of the membrane will determine whether a wearing surface replacement will last 15 years.
2. Extreme traffic or environmental conditions will decrease the useful life of traditional bridges.

3. The substructure can at times outlast the superstructure. The useful life of the substructure should be considered before selecting a rehabilitation alternative.
4. The U.S Army Corps of Engineers document (1997) gives a design life of 50 years for aluminum and plastic pipes. There is evidence that these materials will last much longer.
5. The life of the concrete invert lining is dependent on the longevity of the top plates.
6. The useful life of pipes can vary significantly. Considerations include the cover over the pipe, soil pH and resistivity, presence of salts or other corrosive compounds, plate thickness, and flow velocity.

2.2.7 Cost Comparison for Number of Beams

The following discussion is a guide to compare the cost of reducing the number of beams on steel bridges with full cast in place decks only. Future updates to this procedure will include the use of precast deck panels and the use of precast, prestressed beams. Other issues besides cost must be considered as well when determining the optimal number of beams, such as maintenance of traffic during construction and future maintenance needs (refer to Section 7.3 Economy and Section 2.9.6 Maintainability).

For steel beam bridges with relatively wide decks, the Structural Designer may need to investigate the optimum number of beams to use. Fewer beams will result in less total steel required, but will require more deck concrete, and will have slightly higher fabrication costs per pound of steel. A discussion of the cost comparison method is found here.

Regardless of the number or size of the beams, the raw price of steel supplied from the mill can be considered a constant. For this discussion, we assume a cost of \$0.50/lb. The cost of fabricating, delivering, erecting, and finishing each beam is relatively independent of the weight of the beam, though will be slightly higher for heavier beams due to issues such as additional welding lengths for deeper webs, larger beam surface area that will require more painting, and thicker plates that will require more effort to drill holes. Therefore, one can assume that this cost for the heavier beam will be approximately 10% higher. If significantly more stiffeners will be required for the heavier beam, this number might be even higher. The ratio of costs will then be the number of beams with narrower beam spacing to the adjusted ratio of the number of beams with wide beam spacing.

Wider beam spacing will also require thicker slabs. When slab thicknesses increase appreciably, the support form costs will increase because of the extra

strength required to carry the extra thickness. However, the added support forms cost will be offset by a decrease in labor cost with fewer beams on which blocking must be formed, and also fewer bays in which support forms must be suspended. Therefore, the cost of forming and finishing is assumed to be equal regardless of beam spacing. The price of concrete delivered and placed can be assumed to be equal to about 35% of the unit price of deck concrete. Generally, no cost adjustment is made for reinforcing steel since thicker slabs will have little change in reinforcing steel quantity.

The following example illustrates this method of cost comparison.

Example 2-1 Cost Comparison of Number of Steel Beams

Assume a price comparison of four beams to five beams, with a bid price of \$1.00/lb for five welded beams, and assuming equal stiffeners on all beams. Weight of steel for 5 beams is 30,000 lb.

ratio of beams = $4/5 = 0.80$
 ratio of diaphragms = $3/4 = 0.75$
 assume a cost ratio on fabricating, delivery, and erecting of 0.79, a number chosen between 0.80 and 0.75, but weighted more toward the beam ratio than the diaphragm ratio

5 beams:	mill	\$0.50/lb x 30,000	= \$15,000
	fab/del/erect	\$0.50/lb x 30,000	= <u>\$15,000</u>
			\$30,000
4 beams:	mill	\$0.50/lb x 30,000	= \$15,000
	fab/del/erect	\$0.50/lb x 0.79 x 1.1 x 30,000	= <u>\$13,000</u>
			\$28,000

Assume a bid price of \$450/ yd³ of deck concrete. Assume a five beam bridge will require an 8 inch slab and a four beam bridge will require a 10 inch slab, with quantities of concrete being 150 yd³ and 200 yd³ respectively. The slab costs would be:

8 inch deck:	forming & finishing	\$290 x 150 yd ³ =	\$43,500
	delivery & placing	\$160 x 150 yd ³ =	<u>\$24,000</u>
			\$67,500
10 inch deck:	forming & finishing	\$290 x 200 yd ³ =	\$58,000
	delivery & placing	\$160 x 200 yd ³ =	<u>\$32,000</u>
			\$90,000

Summary:	5 beams:	\$30,000 + \$67,500	= \$97,500
	4 beams:	\$28,000 + 90,000	= \$118,000

2.3 Hydrology, Hydraulics, and Scour

2.3.1 General

Most of Maine's bridges are located over water. Bridge drainage structures will range from large culvert-type structures to multi-million dollar bridges. Although some hydrologic, hydraulic, and scour analysis is necessary for all bridge drainage structures, the extent of such studies should be commensurate with the complexity of the situation, and with the importance of the structure and of the surrounding property.

Minor spans, bridges, and extraordinary bridges are the responsibility of the Bridge Program.

2.3.2 Minor Span/Strut Determination

Designers must determine on a project-by-project basis if a drainage structure is a strut or minor span. A strut is a structure with a span equal to or greater than 5 feet and less than 10 feet. If a structure has a span equal to or greater than 10 feet, or if multiple structures have a combined opening of at least 80 square feet in area, the structure meets the minimum requirements for a minor span. For a minor span or a bridge, the drainage area is typically 2 square miles or larger with a Q50 flow of 500 cfs or larger. The following examples indicate the minimum flow for a pipe, a pipe arch, and a concrete box that meet the definition of a minor span:

- o 10'-3" span by 6'-9" rise steel structural plate pipe arch (18" corner radius) that is 72' long at 0.5% slope with the end mitered to match the slope (inlet control). HW/D is 0.9 or 90% with approximately 325 cfs.
- o 10' diameter steel pipe that is 72' long at 0.5% slope with the end mitered to match the slope (inlet control). HW/D is 0.9 or 90% with approximately 525 cfs.
- o 10' span by 10' rise concrete box culvert that is 72' long at 0.5% slope with square edge headwall and 0° wingwalls (inlet control). HW/D is 0.9 or 90% with approximately 700 cfs.

Table 2-3 can be used for guidance to determine if a structure is a strut or a minor span based upon an approximate flow.

Table 2-3 Design Flow versus Drainage Area and Wetland Percent

Drainage Area (square miles)	Wetland %	Q50 (cfs)
2	1	549
2	5	409
2	10	287
2	14	211
3	1	753
3	5	563
3	10	388
3	15	269
3	18	215

Note: Flows are based on the U.S.G.S. full regression equation. These values are provided for general guidance and should not be used for hydraulic design purposes.

2.3.3 Level of Analysis

2.3.3.1 Level 1 (Qualitative Analysis)

A Level 1 qualitative analysis involves no numerical analysis. It is used for a project when a pipe or pipe arch is being replaced by another pipe in the same location and when the project meets the following criteria:

- No signs of scour or erosion problems
- No reports of flooding problems
- Relatively stable stream (vertically and laterally)
- No history of significant ice jams or debris problems
- No buildings or homes close to the stream
- No reduction in the opening size
- Fish passage is maintained or is not an issue
- Adequate alignment (horizontal and vertical)
- No history of accidents at the bridge location

If the project team decides to use a Level 1 analysis, all the existing records should be reviewed and a site inspection conducted. The site inspection should involve the entire project team. Municipal officials, bridge maintenance, and abutting landowners should be queried for personal

knowledge of flooding activities and all hydraulic and flood information should be documented in the PDR.

2.3.3.2 Level 2 (Basic Analysis)

Most bridge projects fall into the Level 2 basic analysis category. In addition to the qualitative analysis done for Level 1, a numerical analysis is performed for Level 2. Flows are computed, and hydraulics and scour are analyzed for all of the feasible alternatives.

2.3.3.3 Level 3 (Complex Analysis)

Projects that fall into the Level 3 complex analysis category typically have the following concerns:

- Difficulties determining flows (i.e. islands, divided flow, multiple streams merging)
- Uncertainty about the flow angle of attack
- Unstable streams/rivers
- Highly constricted flow with scour problems
- Tidal areas with long bridges
- Project where the opening size may be reduced drastically

Analysis for complex projects may involve a two-dimensional analysis using a program like FESWMS. If there is any uncertainty about what level of analysis applies, the Designer should contact the Bridge Program's hydraulics technical resource people.

2.3.4 Data/Information Collection

The Designer should compile all pertinent information as described below, prior to visiting the site, and before beginning the actual hydrologic analysis for the project. The gathering of such data can simplify the hydrologic analysis and provide the background for good judgment decisions, which may be required.

- o *Topographic survey* - The survey for the project site will be performed by MaineDOT's survey crews or by consultant survey crews as determined by the Survey Coordinator. The plotted survey provides information about the stream's channel and flood plain necessary for the analysis of the structure site. The surveyor's notes and descriptions of the stream and of the existing bridge may provide

valuable information on flood history and for a hydraulic analysis of the site.

- o *Site inspection* - A visit to the project site and to upstream and downstream bridge sites can supply valuable information, such as high water marks on the existing structures or ice markings on trees. Photographs can be taken for reference and to help recall the site conditions. A site inspection can give team the proper perspective of the site conditions, which the survey plan or photographs cannot. If possible, the site inspection should take place after all office records have been gathered.
- o *Inspection reports* - MaineDOT's bridges are inspected at least every two years. Bridges requiring underwater inspections are inspected by divers every five years. These inspection reports should be reviewed for all projects. The underwater inspection report in particular is an excellent source of information about scour problems.
- o *Maintenance reports* - Maintenance reports of work performed on the bridge can provide information on debris, scour, or ice problems that may have occurred. Indications of scour or other problems requiring maintenance work could indicate an undersized structure.
- o *Plans of existing bridges* - The plans of existing bridges at the subject site, as well as at upstream and downstream locations, can give valuable information on flood histories, stream information, and the necessary data for the hydraulic analysis of the structures.
- o *Witnessed observations* - Narrative descriptions of past flood and normal flows may be obtained from Bridge Maintenance Supervisors, Highway Maintenance Supervisors, municipal officials, newspaper accounts, or local residents. Information pertaining to high water elevations at existing bridge sites along with the dates of the occurrences, ice or debris problems, structure adequacy, and other information obtained should be documented.
- o *Aerial photographs* - Aerial photographs can be a helpful tool in evaluating the stream and its flood plain. The Photogrammetry and Control Unit maintains all aerial photograph coverage, of which prints or electronic copies can be made. They may also have aerial photograph contour plans for major highway projects that can also be useful.
- o *Photographs* - Photographs of past flood occurrences can sometimes be obtained from local residents, Bridge Maintenance Supervisors, or in the Bridge Maintenance's photograph files.

- *Stream data from other agencies* - Stream flow and flood related data are sometimes available from other agencies in the State. The major sources are:

U.S. Geological Survey: The U.S.G.S. has numerous gage stations on rivers and streams that collect hydrologic information. Through the use of formulae, this information can be transformed to other locations on the same water course. The Bridge Program's Hydraulic Library has copies of U.S.G.S. annual reports and a computer analysis summary of each gage site, which can be used to determine the existence of a gage location. Real time data from USGS gages is available from the U.S.G.S website (<http://waterdata.usgs.gov/>). If more information is required than can be obtained from these sources, the U.S.G.S. office in Augusta should be contacted.

Natural Resources Conservation Service (NRCS): The NRCS, formerly known as the Soil Conservation Service (SCS), has studies for many flood control projects that contain information on the hydrology and hydraulics of the involved stream. The Hydraulic Library has a location map indicating completed and planned studies. The NRCS office in Bangor should be contacted for detailed information for each site for which information is desired.

Maine Flood Plain Management Program: The Maine Floodplain Management Program has gathered flood information for communities with unnumbered "A" zones on their Flood Insurance Rate Map or Flood Hazard Boundary Map. The information is available on the State of Maine website.

Utilities: Various utility companies have control of many dams in the State, and for most of the larger dams, they maintain flow records and capacity data. The Hydraulic Library has a listing of all known dams in the State with a brief description of the dam and the name of the dam owner.

- *Hydraulic Library* - The Bridge Program's Hydraulic Library has copies of many different Flood Study Reports, such as Corps of Engineer Studies, HUD Flood Insurance Studies, SCS Watershed Studies, and other miscellaneous information pertaining to specific rivers and streams. The Preliminary Engineering Studies and PDRs that have been developed for MaineDOT bridge structures over the years are electronically filed in MaineDOT's TEDOCS document

management system. PDRs with hydrology and hydraulic information are generally available for projects starting in about the year 1975.

- *Local newspapers* - Local newspaper files may have stories on previous floods.
- *Flood insurance studies* - River cross sections used to develop Flood Insurance Rate Maps (FIRM) can be obtained through the Maine Floodplain Management Program in the Department of Economic and Community Development. These cross sections can be used in a hydraulic model such as HEC-RAS. The Bridge Program's Hydraulic Library has paper copies of the FEMA Flood Insurance Studies and Flood Insurance Rate Maps. Flood Insurance Rate Maps can also be viewed / printed on-line as well. If you are interested, the Maine Flood Plain Management Program web site has some instructions posted to help you through this process.

All of the above sources of information may provide valuable assistance and supplementary information that can be used advantageously; however, discrepancies sometimes are revealed when these data are compared. This indicates the need for verification and proper evaluation of the flood data, regardless of the source.

2.3.5 Vertical Datum

Since January 2000, all new projects, with a few exceptions, are referenced to the North American Vertical Datum (NAVD) of 1988.

Commentary: If there is any doubt about which vertical datum was used for a project, please contact the Survey Coordinator.

Many of MaineDOT's existing plans, existing flood studies, historical flood information, and U.S.G.S. topographic maps are based on the National Geodetic Vertical Datum (NGVD) of 1929. The elevations based on this older datum must be converted to the newer NAVD of 1988. The elevations are adjusted using the following equation:

$$\text{Elevation xxx.xxx (NGVD 1929) - datum shift} = \text{Elevation xxx.xxx (NAVD 1988)}$$

The datum shift ranges between 0.591 feet and 0.722 feet. The exact datum shift for a specific location in Maine can be found at the following website:

http://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl

The following data must be entered on the web page:

CHAPTER 2 – PRELIMINARY DESIGN

- North Latitude (required)
- West Longitude (required)
- Orthometric Height (optional)

Latitude and Longitude may be entered in any of the following three formats, including blank spaces:

Degrees, minutes, and decimal seconds (xxx xx xx.xxx)

Degrees and decimal minutes (xxx xx.xxx)

Decimal degrees (xxx.xxxxx)

The following example illustrates how to apply the datum shift:

Example 2-2 Datum Shift

This information comes from the Gouldville Bridge in Presque Isle.
Q100 Elevation = 431' from Flood Insurance Study based on (NGVD 1929).

Step 1: Go to website and get datum shift by entering latitude and longitude for the location you are interested in.

Latitude = 46.667⁰
Longitude = 68.00⁰
Datum shift = 0.627'

Step 2: Subtract datum shift (i.e. correction factor) from elevation based on NGVD 1929 to convert to NAVD 1988.

$$(\text{NGVD 1929}) - (\text{correction}) = (\text{NAVD 1988})$$

$$431' - 0.627' = 430.373'$$

Hydrology, hydraulics, and scour reports should state which vertical datum is used. For example, the following statement can be added at the end of any report:

Note: All elevations based on North American Vertical Datum (NAVD) 1988. Elevations based on the National Geodetic Vertical Datum (NGVD) 1929 were converted to NAVD by the appropriate shift (0.627') using the NGS Vertcon program.

2.3.6 Tidal Elevation Computations

Full daily tide predictions are limited to a small number of reference stations. Maine has only three reference stations in Eastport, Bar Harbor, and Portland. Tide predictions at other locations are referred to as "subordinate stations", can be obtained by applying specific differences to the daily tide predictions for one of the reference stations. The application of time differences and height ratios will generally provide reasonably accurate approximations at subordinate stations, however, they cannot result in predictions as accurate as those listed for the reference stations.

The National Oceanic and Atmospheric Administration, National Ocean Service (NOS) is in the process of updating the nation's tidal datums to a new National Tidal Datum Epoch (NTDE) from 1983 to 2001 to reflect changes in mean sea level along the nation's coast. The new NTDE will provide up-to-date tidal datum information. Whenever possible, data from the 1983-2001 NTDE should be used when computing elevations. The NTDE is a specific 19-year period over which tide observations are taken to determine Mean Sea Level and other tidal datums such as Mean Lower Low Water and Mean High Water. This latest update will define the 19-year period as 1983-2001. The 19-year period includes an 18.6 year astronomical cycle that accounts for all significant variations in the moon and sun that cause slowly varying changes

in the range of tide. The following examples show how to determine tidal elevations at a reference station and at a subordinate station.

Example 2-3 Tidal Elevation at Reference Station

Determine the following elevations for the Eastport, Maine reference station:

- Highest Observed Water Level
- Mean Lower Low Water (MLLW)
- Mean Low Water (MLW)
- Mean Tide Level (MTL)
- Mean High Water (MHW)
- Mean Higher High Water (MHHW)
- Lowest Observed Water Level
- Predicted High Tide Elevation for 2003

Step 1: Obtain the tidal datum information from the tidal gage site on the NOAA website (<http://tidesandcurrents.noaa.gov>). Use the menus at the top of the website or search to find the data for the Eastport tide gauge.

About two thirds of the way down the web page for Eastport, you will find the tidal datums section for the particular site. For example, the tidal datums section will look like the following for 8410140 EASTPORT, PASSAMAQUODDY BAY:

TIDAL DATUMS

Tidal datums at EASTPORT, PASSAMAQUODDY BAY based on:

LENGTH OF SERIES: 19 Years
 TIME PERIOD: January 1983-December 2001
 TIDAL EPOCH: 1983-2001
 CONTROL TIDE STATION:

Elevations of tidal datums refer to Mean Lower Low Water (MLLW), in METERS:

HIGHEST OBSERVED WATER LEVEL (01/10/1997)	= 7.383
MEAN HIGHER HIGH WATER (MHHW)	= 5.844
MEAN HIGH WATER (MHW)	= 5.729
NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)	= 3.029
MEAN SEA LEVEL (MSL)	= 2.958
MEAN TIDE LEVEL (MTL)	= 2.932
MEAN LOW WATER (MLW)	= 0.136
MEAN LOWER LOW WATER (MLLW)	= 0.000
LOWEST OBSERVED WATER LEVEL (08/09/1972)	= -1.426

Step 2: Convert the tidal datum information to the correct vertical datum. The tide information needs to be converted to the NAVD. MaineDOT has been surveying using the NAVD since about the year 2000.

Highest Observed Water Level (01/10/1997):
 $7.383 \text{ m} - 3.029 \text{ m} = 4.354 \text{ m}$

MHHW: $5.844 \text{ m} - 3.029 \text{ m} = 2.815 \text{ m}$

MHW: $5.729 \text{ m} - 3.029 \text{ m} = 2.700 \text{ m}$

CHAPTER 2 – PRELIMINARY DESIGN

NAVD -1988: $3.029 \text{ m} - 3.029 \text{ m} = 0.000 \text{ m}$
MSL: $2.958 \text{ m} - 3.029 \text{ m} = -0.071 \text{ m}$
MTL: $2.932 \text{ m} - 3.029 \text{ m} = -0.097 \text{ m}$
MLW: $0.136 \text{ m} - 3.029 \text{ m} = -2.893 \text{ m}$
MLLW: $0.000 \text{ m} - 3.029 \text{ m} = -3.029 \text{ m}$

Lowest Observed Water Level (08/09/1972):
 $-1.426 \text{ m} - 3.029 \text{ m} = -4.455 \text{ m}$

Step 3: Convert elevations from meters to feet. Tidal datum information based on the NTDE from 1983 -2001 is in meters.

Highest Observed Water Level (01/10/1997)
 $4.354 \text{ m} \times 3.2808 \text{ ft/m} = 14.285 \text{ ft}$

MHHW: $2.815 \text{ m} \times 3.2808 \text{ ft/m} = 9.236 \text{ ft}$
MHW: $2.700 \text{ m} \times 3.2808 \text{ ft/m} = 8.858 \text{ ft}$
NAVD -1988: $0.000 \text{ m} \times 3.2808 \text{ ft/m} = 0.000 \text{ ft}$
MSL: $-0.071 \text{ m} \times 3.2808 \text{ ft/m} = -0.233 \text{ ft}$
MTL: $-0.097 \text{ m} \times 3.2808 \text{ ft/m} = -0.318 \text{ ft}$
MLW: $-2.893 \text{ m} \times 3.2808 \text{ ft/m} = -9.491 \text{ ft}$
MLLW: $-3.029 \text{ m} \times 3.2808 \text{ ft/m} = -9.938 \text{ ft}$

Lowest Observed Water Level (08/09/1972):
 $-4.455 \text{ m} \times 3.2808 \text{ ft/m} = -14.616 \text{ ft}$

Step 4: Determine the highest predicted tide for the current year.

Go to the NOAA Tides and Currents website and find the Eastport tide gauge data.

Review the Tides/Water Levels data for the entire year and find the date with largest height.

April 19, 2003 12:09 am 22.3 ft (datum is MLLW)

2003 predicted high tide = $-9.938 \text{ ft (MLLW)} + 22.3 \text{ ft} = 12.362 \text{ ft}$

Example 2-4 Tidal Elevation at Subordinate Station

Determine the following elevations at West Quoddy Head using Eastport as the reference station.

MLLW
MLW
MTL
MHW
MHHW
Predicted High Tide Elevation for 2003

Step 1 through **Step 4**: See Example 2-3 for the Eastport location.

Step 5: Obtain the values for the mean range, spring range, and MTL for the West Quoddy Head location (subordinate station) from the NOAA Tides and Currents website.

West Quoddy Head
Mean range = 15.7 ft
Spring range = 17.9 ft
MTL = 8.2 ft

Step 6: Compute tide levels at West Quoddy Head

MTL Eastport = MTL West Quoddy Head

MHW West Quoddy Head = MTL Eastport + Mean Range @ West Quoddy Head/2
 $-0.318 \text{ ft} + 15.7 \text{ ft}/2 = 7.5 \text{ ft}$

MLW West Quoddy Head = MTL Eastport - Mean Range @ West Quoddy Head/2
 $-0.318 \text{ ft} - 15.7 \text{ ft}/2 = -8.2 \text{ ft}$

MLLW West Quoddy Head = MTL Eastport - Mean Tide Level @ West Quoddy Head
 $-0.318 \text{ ft} - 8.2 \text{ ft} = -8.5 \text{ ft}$

MHHW West Quoddy Head = MLLW @ West Quoddy Head + Spring Range @ West Quoddy Head
 $-8.5 \text{ ft} + 17.9 \text{ ft} = 9.4 \text{ ft}$

Step 7: Determine the highest predicted tide for the current year at West Quoddy Head.

On the NOAA Tides and Currents website, find the Eastport tide gauge, which is the closest reference station. Review the data for the entire year and find the date with largest height.

April 19, 2003 12:09 am 22.3 ft (datum is MLLW)

Get the following reference from the MaineDOT Library:

Tide Tables 2003, High and Low Water Predictions, East Coast of North and South America including Greenland

In Table 2 of the Tide Tables book under West Quoddy Head, find the ratio of height differences at high water.

West Quoddy Head Ratio = 0.86

$0.86 \times 22.3 \text{ ft} = 19.17 \text{ ft}$ (datum is MLLW)

2003 predicted high tide = $-8.5 \text{ ft (MLLW)} + 19.17 \text{ ft} = 10.7 \text{ ft}$

2.3.7 *Changes in Sea Level*

Historical data from NOAA shows that the sea level along the Maine coast over the past 80-100 years has risen between 0.5 and 0.75 feet per 100 years relative to local datums. More detailed information is available from the NOAA Tides and Currents website in the Sea Level Trends section. Based on this historical data and NOAA projections, the proposed design should assume 4 feet of sea level rise per 100 years.

2.3.8 *Documentation*

The PDR includes a hydrology, hydraulics, and scour report and backup information. Backup information should include, but is not limited to, the following: computer printouts (input and output), drainage area map, hydrology computations, hydraulic computations, scour computations, and eyewitness reports about flooding.

The PDR is the main source of hydrologic, hydraulic, and scour information for a bridge project. If there are any changes made to the project after the PDR has been completed that impacts hydrology, hydraulics, and/or scour, it should be documented and included in the PDR as an addendum.

It is often helpful and sometimes necessary to refer to plans, hydrology, hydraulic, and scour analyses long after the actual construction is completed. They can be useful in the analysis of an upstream or downstream structure, in the future replacement of the structure, or in the evaluation of the hydraulic performance of the structure after large floods. Documentation provides a quick reference and a construction aid for the Contractor and the Resident in the construction of a bridge structure. This information is also helpful to other state agencies such as Floodplain Management, as a source of best available data for Q100 elevation when a formal flood study has not been done for a river.

2.3.9 *Hydrology*

2.3.9.1 Introduction

Hydrologic analysis is a very important step prior to the hydraulic design of a bridge drainage structure. Such an analysis is necessary for determining the flow that the structure will be required to accommodate. The flow, or

discharge, is a hydraulic "load" on the structure and the determination of its magnitude is as important as the determination of proper structural loads. These guidelines give a recommended approach to the hydrologic analysis of bridge drainage structures. The guidelines are not all-inclusive, nor are they intended to require strict compliance, but they are presented as a guide. Hydrology is not an exact science, and it requires the use of good engineering judgment to evaluate the available information and arrive at logical and suitable conclusions.

2.3.9.2 Discharge Rate Policy

The following discharge rates need to be computed for the hydraulic design of bridges and minor spans:

- Q1.1 – ordinary high water (OHW) discharge
- Q50 - design discharge
- Q100 or flood of record - check discharge

Other discharge rates may need to be computed as follows:

- Flows less than Q1.1 - discharges used to check for fish passage in culvert-type structures
- Q10 - discharge used in designing temporary bridges
- Q500 - discharge used in evaluating scour

The determination of the design and check discharges are accomplished through the application of one or more discharge formulae given in this text, combined with the information obtained through information sources and/or through hydraulic analysis of existing structures. Discharge adjustment factors are found in Appendix C Hydrology/Hydraulics.

2.3.9.3 Discharge Formulae

Drainage studies for most projects are requested from the Hydrology Unit in the Environmental Office. The unit provides the Designer with a spreadsheet based upon the U.S.G.S. full regression equations discussed in Appendix C Hydrology/Hydraulics, and Section 2.3.9.4 Rural Watersheds, which follows. Unless gaged data is applicable to the project, dams are present on the section of waterway of interest, or if the U.S.G.S. full regression equation is not applicable, the spreadsheet provided is all that is required for hydrologic analysis. For cases where the spreadsheet provided by the Hydrology Unit is not adequate, refer to the following Sections 2.3.9.4 through 2.3.9.4B.

2.3.9.4 Rural Watersheds

Most watersheds for bridges in Maine are rural in nature. A rural area can generally be defined as one having a high percentage of woods, mixed cover, or fields, and is essentially an undeveloped area with respect to commercial sites and residences. The best source of flow data for rural watersheds is gaged data from the U.S.G.S. gaging station network. Methods for transposing gaged data are including on the following pages. If gaged data is not available, the U.S.G.S. full regression equation can be used. Appendix C contains this equation, as well as a hydrology tabulation form for use with the equation. The report that explains the 1999 U.S.G.S. full regression equation is “Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals” by Glenn A. Hodgkins, published by U.S.G.S. in 1999 and available from their website.

A. Urban Watersheds

The U.S.G.S. full regression equation does not apply to urbanized drainage basins or small drainage basins that may experience future development and land use changes. An urban area can generally be defined as one having a very low percentage of woods, mixed cover, or fields, and is essentially a developed area with commercial sites and residences. Potential future development in the watershed should be considered when determining the design flow.

The following methods can be used for small, urbanized drainage basins:

Size of Drainage Area	Hydrologic Method
Greater than 3200 acres	NRCS TR-20 or HEC-1 Method
Greater than 20 acres	Sauer and others (1983)

NRCS TR-20 and HEC-1 Methods are explained in the “Urban & Arterial Highway Design Guide.” Sauer and others (1983) is an urban regression equation (Hodgkins, 1999).

B. Hydraulic Analysis

Flows based on observed and recorded high waters at or near bridges may be determined by performing a hydraulic analysis using the methods discussed in 2.3.10.2 Hydraulic Analysis. For culverts, Bodhaine, 1968, can be used.

All of the applicable methods that may be used for the watershed in question should be utilized. However, large variations in answers may

result. Knowledge of the limitations and accuracies of each method, coupled with other sources of information and good engineering judgment will be necessary to arrive at a reasonable selection of discharge values.

2.3.10 Hydraulics

2.3.10.1 Introduction

A major aspect in highway design and construction is the crossing of streams and rivers. A concurrent problem is the encroachment of the highway on the flood plain, or even the stream channel. The design of the crossing must be made to insure the safety of the traveler, must protect the river environment, must not create hazards or problems to adjacent landowners and the community, and must be economical. Good engineering judgment combined with knowledge of hydrology and hydraulic sciences is required to determine the design of river crossings.

At most sites, several factors affecting the roadway grade and hydraulic opening need to be considered. These factors generally fall into two categories:

Impacts

- Property impacts
- Wetland impacts
- Historical or archaeological impacts
- Marine traffic
- Constructability
- Cost

Risk

- Importance of the roadway
 - Corridor Priority
 - MEMA Evacuation Route
- Remaining bridge service life
- Accessibility of the bridge during flood events
- Feasible detour routes during flood events

There may be instances where meeting the minimum design criteria is unreasonable due to these factors. If the minimum design criteria is not met with the proposed design, the PDR should state and discuss the difference. In all cases, the reasoning and factors involved in the design process should be clearly documented in the PDR.

At no time should the design criteria be considered a substitute for careful evaluation of site specific factors and good engineering judgment. The level of risk allowed will vary based on site conditions. At some sites, maintaining clearances will be paramount regardless of the impacts; at others, accepting a higher level of risk may be the appropriate option.

Bridges in Maine are designed for both riverine and tidal stream crossings. Riverine bridges are designed for steady flow at the peak discharge for the design storm. Hydraulics design for riverine bridges establishes:

- Minimum finished grades
- Bridge location
- Bridge length
- Span lengths
- Orientation of substructure
- Foundation requirements through scour analysis

Tidal bridges are designed for unsteady flow conditions during the complete rise and fall cycle of the tide. Hydraulic design for tidal bridges establishes the minimum finished grade and minimum depth requirements for the foundation through scour analysis. For special cases, other features may require hydraulic design. For sites further upstream, riverine flow becomes dominant. In some cases both riverine and tidal flow must be analyzed to determine the controlling flow at a bridge.

2.3.10.2 Hydraulic Analysis

The depth or extent of the hydraulic analysis for a bridge structure should be commensurate with the cost and complexity of the project and the problems anticipated.

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The main tools for the hydraulic analysis of bridge structures are as indicated below. Additional analysis methods may be used as deemed necessary.

Culvert-type structures:

- Design charts from HDS No. 5, 1985
- HY 8 Culvert design and analysis program by FHWA (Part of Hydrain program)
- Principles of open channel hydraulics
- Other commercially available software programs

Bridges:

- The Army Corp of Engineers program HEC-RAS (preferred program)
- The U.S.G.S. Computer Program "WSPRO"
- Principles of open channel hydraulics

A. Structure Capacity (Riverine)

All bridges and minor spans should be designed for Q50 with the following constraints:

Culvert-type structures - The headwater depth versus structure depth ratio (HW/D) should be approximately equal to or less than 0.9. For twin pipes or pipe arches, the HW/D ratio should be less than 0.9. A minimum of 1 foot of freeboard at the edge of the pavement at Q100 or the flood of record is preferred when outlet conditions control.

Major riverine bridges - A freeboard depth of 4 feet minimum between the bottom of the superstructure and the backwater elevation should be provided on major river crossings. As much as 10 feet of freeboard depth should be provided when practical.

Other riverine bridges - A depth of 2 feet minimum is recommended on smaller streams where there has been no history of ice jams.

All bridge-type structures should also be capable of passing the Q100, or the flood of record, whichever is greater, without any serious harm to the structure, roadway, or adjacent property. This may be accomplished by allowing an overtopping of the approaches if the structure cannot be reasonably sized to accommodate the flow, with the approval of the Engineer of Design. When possible, there should be 1 foot of freeboard at Q100.

Occasionally, freeboard depths may need to be increased for high waters caused by some occurrence other than the design flow, such as for an ice jam, the collapse of a dam, or some future construction that may affect the depth of flowage.

B. Structure Capacity (Tidal)

Culvert-type structures in tidal area - The headwater depth versus structure depth (HW/D) ratio should be equal to or less than 0.9 at Q50 with flow at MHW under inlet control conditions.

Bridges in tidal area - Bridges on tidal rivers/streams should be designed to protect the bridge structure itself. Most of the surrounding land and the approach roadways may be inundated by relatively frequent tidal storm surges. The minimum design freeboard in these areas is 2 feet above Q10 (based on MHW with sea level rise), including wave heights.

C. Analysis Types in Tidal Areas

- *Qualitative analysis:* This method can be used if the criteria in Section 2.3.3 Level of Analysis are met, and if the team has decided to use the simplified approach.

- *Steady flow:* This type of analysis checks at least two or more points in the entire tide cycle. Typically the following cases would be investigated:

Case 1: Q50 flow with tailwater at mean high water (MHW): This case typically determines the size of the opening and the bottom of beam elevation.

Case 2: Q50 flow with tailwater at mean low water (MLW): This case typically results in the highest velocities. The velocity is used to design erosion and scour measures.

- *Unsteady flow:* This type of analysis checks the entire tide cycle at 15 minute intervals over a 48 hour period. The typical cases that would be analyzed include the following:

Case 1: Typical everyday tides with low upland flow (used to verify the model).

Downstream boundary condition - Typical tide cycle based on mean tide range

Upstream boundary conditions - Constant Q1.1 flow or a lower more typical flow

Case 2: High upland flows with no coastal storm.

Downstream boundary condition - Typical tide cycle based on mean tide range

Upstream boundary conditions - Constant Q50 flow

Case 3: Late summer/early fall hurricane with low upland flow.

Downstream boundary condition - Typical tide cycle based on mean tide range with storm surge due to a Category 1 hurricane. A Category 1 hurricane equates to about a 50 year storm surge. The peak of the storm surge should be checked for the following four different times:

1. Peak of storm surge at mid rising tide
2. Peak of storm surge at high tide
3. Peak of storm surge at mid falling tide
4. Peak of storm surge at low tide

Upstream boundary condition - Constant Q1.1 flow or a lower more typical flow.

An unsteady flow analysis in a tidal area requires cross sections (for 1-D analysis using HEC-RAS) and/or a digital terrain model (DTM, for 2-D analysis) that covers at least 90% of the area within the drainage basin affected by the tides. Getting the survey information to create the hydraulic model for an unsteady flow model is difficult and expensive.

2.3.10.3 Discharge Velocities

The velocity at the outlet or downstream side of a bridge structure can be a controlling feature of the structure opening. The scour susceptibility of the stream and scour protection measures should be a major consideration in the sizing of a bridge. The velocity through the existing bridge and the scour conditions should be evaluated. If the present conditions do not show any cause for scour concern, the same velocities may be used in the design of a new structure. Higher velocities may be allowed if the site evaluation determines those velocities will not be detrimental.

2.3.10.4 Backwater

A bridge is generally an obstruction in a stream or river that can cause a rise in water level behind the bridge, known as backwater. The height of this backwater can also be a controlling factor in the sizing of a bridge. The affect of backwater on upstream property must be considered. The determination of water levels from an existing bridge is an important guide in evaluating the backwater height of a new structure. FEMA regulations require that the backwater at Q100 increase no more than 1 foot.

2.3.10.5 Dams

Bridges influenced by the presence of dams should be analyzed hydraulically for the following two situations:

- Existing dam remains in place
- Existing dam is removed

Many dams throughout Maine are now being removed. All new bridges should be designed so that any nearby dams can be removed with no adverse effect to the bridge. Some analysis may be needed for the case where a major dam (typical high head) will remain in place. The water level may be lowered for dam maintenance or emergencies for an extended period of time.

2.3.10.6 Fish Passage

Designers should refer to the latest guidance from MaineDOT's Environmental Office to ensure that fish passage is maintained.

2.3.11 Scour

Commentary: Flooding is the most common cause of bridge failure, with the scouring of bridge foundations being the most common failure mechanism. The catastrophic collapse of the Interstate 90 crossing of Schoharie Creek near Amsterdam, NY on April 5, 1987, is one of the most severe bridge failures in the U.S. Two spans fell into the water after a pier supporting the spans was undermined by scour. Five vehicles plunged into the creek killing 10 people. The National Transportation Safety Board concluded that the bridge footings were vulnerable to scour because of inadequate riprap around the base of the piers and a relatively shallow foundation. The I-90 collapse focused national attention on the vulnerability of bridges to failure from scour and resulted in revisions to design, maintenance, and inspection guidelines.

MaineDOT initiated a scour-screening program in 1987 in response to FHWA Technical Advisory TA 5140.20 (succeeded by TA 5140.21 and TA 5140.23). The advisories ultimately require that a master list be generated of all bridges that require underwater inspection, and that all applicable bridge foundations be evaluated and prioritized according to their vulnerability to scour damage. Reliable equations to compute local scour depths are available for piers. A report by the USGS titled "Observed and Predicted Pier Scour in Maine" is available from their website. The report confirms that the local pier scour predicted by the latest version of the CSU equation in the Hydraulic Engineering Circular 18 Fourth Edition May 2001 on page 6.2 are reasonable.

2.3.11.1 New Bridges

Bridges over waterways with scourable beds should be designed to withstand the effects of scour from a superflood (a flood exceeding Q100) without experiencing foundation movement of a magnitude that requires corrective action. A scour analysis will be performed for all bridge-type structures using the methods in the latest version of HEC-18. The design flood for scour is the lesser of Q100 or the overtopping flood. Maximum scour depths will be produced by the overtopping flood. Scour should also be computed for the superflood, defined as Q500 or the overtopping flood if it is between Q100 and Q500. Q500 can be estimated as 1.18 times the magnitude of the Q100, if Q500 cannot be computed by other means.

The bridge foundation should be designed for the normal factor of safety as specified in AASHTO Standard Specifications below the scour depths estimated for Q100. The bridge foundation should have a factor of safety of 1.0 for scour produced by the superflood. The footings should be placed a minimum of 2 feet below the design flood scour level. Where pile bents are used, the design friction or point bearing should be achieved below the

depth of the design scour. There must be sufficient pile penetration below the scour line to provide lateral stability and structural capacity to support the calculated loads.

The geotechnical analysis of bridge foundations should be performed on the basis that all stream bed material in the scour prism above the total scour line for the scour design flood has been removed and is not available for bearing or lateral support.

When analyzing piers for local scour, the pier width should be increased by a minimum of 25% to account for the collection of debris.

The bottom of spread footings on soil for nonspill-through type abutments shall be located a minimum of 6 feet below the lowest streambed elevation in the immediate vicinity of the bridge (two bridge lengths upstream or downstream of the bridge or 50 feet, whichever is larger).

2.3.11.2 Existing Bridges

If there is a history of scour at an existing bridge that is to be rehabilitated, then a scour evaluation should be performed for the following project scopes to determine whether the bridge is scour-critical:

- Deck Replacement
- Superstructure Replacement
- Bridge Widening

A scour-critical bridge is one with abutment or pier foundations that are rated as unstable due to one of the following:

- Observed scour at the bridge site
- Scour potential as determined from a scour evaluation study (refer to HEC-18 Chapter 5)

Designers should consult with Bridge Maintenance on scour-critical bridges to determine if the use of non-designed countermeasures and/or regular inspections may be an acceptable method to reduce the risk of failure. If not feasible, a hydraulic analysis will be needed to properly design scour countermeasures or to analyze a new bridge structure.

A plain riprap apron can be used as a designed scour countermeasure around an existing pier, if the velocity at the design flow is less than 5.3 fps. A heavy riprap apron can be used as a designed scour countermeasure around an existing pier if the velocity at the design flow is greater than 5.3 fps, but less than 8.8 fps. The riprap apron should have a minimum width of 10 feet perpendicular to the centerline of the structure.

2.3.11.3 Riprap Slope Protection

Riprap slope protection should normally be plain riprap and be 3 feet thick with the toe constructed 1 foot below final ground or streambed elevation. Thicker riprap and/or deeper toe depths may be warranted at locations of severe stream velocities and/or in scour susceptible streambeds, as determined by hydraulic analysis. When heavier riprap protection is needed, it should be a 4 foot thick layer of heavy riprap with the toe constructed 3 feet below final ground or streambed elevation.

Bedding material, which will also serve as a filter blanket, should be provided beneath all riprap installations. In tidal locations, a geotextile filter material should be utilized under the riprap instead of the bedding material.

On stream crossing projects, riprap should be placed in front of spill through type abutments and wingwalls. The top of the riprap should be located to protect the abutment embankment from scour and to provide adequate cover above the bottom of footings in accordance with this section and Chapter 5 Substructure.

For culvert-type structures, riprap should be placed on the embankment slopes around the upstream and downstream ends of the structure. The top of the riprap should be located at the Q50 elevation. The Q50 elevation may be lower on the downstream end due to stream slope and/or upstream ponding as determined by the hydraulic analysis of the site and structure. The riprap should extend horizontally a minimum of 5 feet on either side of the culvert.

Scour and/or erosion protection of stream channel bottoms at the inlet and/or outlet of culvert-type structures should be provided where required to prevent scouring of the streambed and undermining of the structure. It should be designated as a plain riprap apron and be 2 feet thick. Culverts with high outlet velocities may require a 3 foot thick heavy riprap apron. Culverts with very high outlet velocities may need energy dissipators. Energy dissipators should be designed in general accordance with the procedures in FHWA HEC No. 14.

Riprap should also be provided on the roadway approach embankments of bridge and culvert-type structures to further protect the structure from erosion or scour damage. The lateral extent of riprap protection of the embankments from a bridge or culvert-type structure should be sufficient to provide protection from unimpeded flow upon the embankment slopes on the upstream side of the stream crossing, and for a distance of 5 feet beyond the top of stream banks on the downstream side of the stream crossing. The top of the riprap should be located at the Q50 elevation.

Protection of roadway embankments, other than by vegetative cover, is generally not necessary except at locations where a stream crossing is on a severe skew to the flood plain, and stream flow can occur along the embankment slopes.

At tidal crossings, the top of riprap should be located at a minimum elevation of 2 feet above MHHW. Consideration should be given to placing the riprap even higher due to waves and wave runup. Each site should be evaluated on a project-by-project basis.

Bridges located immediately on the ocean should use heavy riprap. Heavy riprap should also be used when the average velocity is 15 fps or greater. The use of heavy riprap should be given serious consideration when the average velocity is between 12 fps and 15 fps, especially when ice is a problem.

2.4 Maintenance of Traffic During Construction

2.4.1 General

The method of maintaining traffic during construction must be considered for all bridge projects. In general, the preferred method is to close the bridge and detour traffic on adjacent roads. This will usually result in the shortest construction time, and therefore, a less expensive project. However, this method is not always feasible due to long detour routes, poor quality roads, or high traffic volumes.

The following factors should be considered when determining the best method of maintaining traffic.

- o *Traffic composition.* A high percentage of trucks, RV's, or school buses will require larger turning radii and wider lanes.
- o *Mobile homes and other wide loads.* On projects where staged construction is required for extended periods of time on single access roads (only one way in and out) consideration should be given to coordinating the movement of mobile homes and other wide loads. This can be done by either coordination with the Contractor during construction, requiring the Contractor to open the bridge on preset days in the contract documents, or maintaining at least one 16 foot or wider lane during construction.
- o *Traffic volume.* One lane can accommodate up to 1700 vehicles per hour in free flow conditions. Low volumes can be more easily absorbed on local roads.

- o *Proposed lane width.* Eleven feet is the minimum width required, though 10 feet may be used in special circumstances. For high volume roads or roads with many trucks, lanes should be 12 feet wide or greater.
- o *Required work zones.* Sufficient width must be provided for the Contractor to accomplish the scope of work.
- o *Bridge length.* A bridge greater than 500 feet in length may cause unacceptable stop times when using alternating one-way traffic. Shorter work zones should be considered.
- o *Adjacent side roads or driveways.* Provisions should be made to allow traffic to enter and exit.
- o *Emergency vehicles.* The effect of construction on response time of police, fire, and ambulances must be considered.
- o *Geometric issues.* Advanced warning devices may be needed if visibility is compromised as the driver approaches.
- o *Pedestrian and bicycle traffic.* A determination should be made whether pedestrian and bicycle traffic can be maintained during construction, and how it will be done.
- o *Bridge curvature.* A curved bridge may have less usable width, and will likely require wider lanes.

A Traffic Control Plan (TCP) must be developed for every project. Responsibility for this plan is with either the Contractor or MaineDOT, as determined at the PS&E stage. The complexity of the project may steer the Structural Designer toward keeping this responsibility within MaineDOT, to assure compliance with the conceptual design. Any TCP must comply with the latest edition of the Manual of Uniform Traffic Control Devices (MUTCD).

2.4.2 *Methods to Maintain Traffic*

There are three ways commonly used to maintain traffic. They are discussed here in order of generally increasing costs. The fourth method is an innovative approach that has been used successfully on a number of projects.

2.4.2.1 Close the Road and Detour on Existing Roads

Care should be taken in evaluating proposed detour routes. Detours should be routed using state or state aid highways with input from both the Division Traffic Engineer and municipal officials. Exceptions to using these highways can be made with written concurrence of the town, with

agreement to relieve MaineDOT of responsibility for any deterioration caused by the detoured traffic. It is prudent to discuss the detour with emergency services prior to advertising.

2.4.2.2 Staged Construction

This involves maintaining traffic on part of the existing bridge for the first phase of construction, building a portion of the new bridge, and then moving traffic to the new portion to complete demolition of the existing and construction of the new structure. If possible, two lanes of opposing traffic should be maintained during staged construction. If only one lane is maintained, alternating one-way traffic can be controlled either by using temporary signals, or by posting with a yield/stop condition. Yield/stop conditions may be considered if the average annual daily traffic (AADT) is less than 1500 vehicles per day, and the sight distance is adequate for the posted speed or the 85th percentile speed.

2.4.2.3 Temporary Bridge

A temporary bridge should be considered when other methods are not feasible. Depending on expected traffic volumes, the temporary bridge may carry one lane of alternating one-way traffic, or two lanes of opposing traffic. The Contractor is responsible for the design of the bridge, with approval obtained by MaineDOT. Sufficient right-of-way and environmental permitting must be obtained to allow the Contractor to design the structure adequately. Prior to construction, the Resident should carefully review the Contractor proposed design and drawings of the temporary bridge to assure compliance with Standard Specifications Section 510 – Special Detours. The Contractor proposed design must be within the right-of-way provided and the obtainment of additional right-of-way by the Contractor will not be allowed. The Structural Designer may be asked to review the Contractor's plans and computations.

2.4.2.4 Innovative Methods

The existing superstructure can sometimes be used to maintain traffic off the existing alignment at a significant savings over a temporary bridge. Temporary supports can be constructed, and the existing superstructure slid over to rest on the temporary supports. This has been done with both truss structures and conventional girder/deck systems. The proposed bridge is then constructed either in whole or using staged construction methods, while traffic is maintained on the existing superstructure.

When night work can be specified, wearing surface replacement on high volume bridges has been done using rapid construction methods, such as grinding the wearing surface and replacing it with a fast-setting topping.

The work is done in sections over several nights, keeping one lane open, with the bridge reopened to two lanes of traffic by morning each day.

For work on Interstate bridges, the use of crossovers has been incorporated on large deck replacement projects. Crossovers are constructed on both ends of the bridge allowing for two-way travel on one side of the divided highway and closure of the other side. This scheme has also been used for the construction of new overpass bridges.

2.5 Geotechnical and Survey

Prior to the start of field work, the team should agree upon the necessary field data. The Structural Designer may meet with the Survey Coordinator and the Geotechnical Designer to determine the limits of survey and optimal locations for test borings, respectively.

2.5.1 Geotechnical

Geotechnical design must be done in conjunction with structural design to optimize the selected structure type for the PDR. The Structural Designer and Geotechnical Designer will work together as part of the team process. Considerations include:

- o The Geotechnical Designer will provide preliminary foundation and earthwork design recommendations for the PDR. This preliminary analysis may require a subsurface exploration, or may be done based upon existing subsurface data.
- o Test borings will generally be required for each proposed substructure unit for final design. Precise boring locations cannot be determined until the Structural Designer has set the proposed alignment with stations for abutments and piers.
- o Reuse of existing substructure units will usually require an analysis of the substructure stability under new loads. Refer to Sections 10.6 and 10.7 for information regarding substructure rehabilitation and substructure reuse, respectively.

2.5.2 Field Survey

Survey of the bridge site will be necessary for most projects (refer to Section 1.5 Small Bridge Initiative for exceptions). Ideally, the Designer should meet with the Survey Coordinator, preferably on site, to determine the limits of survey. However, many times the survey is done prior to the Structural Designer beginning work in order to advance the project schedule.

The “Survey Manual” gives guidelines used by survey crews to obtain project survey. The most discriminating characteristic is whether the project is a replacement or rehabilitation. For a replacement project, survey will tie in the structure by locating the corners. If accurate as-built plans are available, this will often be enough information to design the new structure. For a rehabilitation project, highly detailed structure information is necessary. For example, the information gathered will include curb lines, wingwalls top and bottom, breast walls, bridge seats, piers top and bottom, etc.

The following information is collected routinely on a bridge project:

- o *Limits of survey along the roadway:* Most projects will require at least 150 to 200 feet on either end of the bridge to accommodate required guardrail lengths. If the roadway is curved, consider the need to match into the existing curve and obtain enough data points to do so. If the new structure is expected to be off alignment, additional length will be needed.
- o *Limits of survey from the existing centerline:* Most projects will require at least 60 feet from the centerline, to accommodate toes of slope and to define drainage.
- o *Stream data:* The edge of stream for 75 feet upstream and downstream will be obtained for right-of-way purposes. Bottom of stream points will be obtained 60 feet from the centerline, usually by wading or from a small boat. For larger structures, a string will be obtained at a distance of 2 times the span length upstream and 1 times the span length downstream for hydraulic analysis. Additional sections should be requested, if needed.
- o *Wetlands:* This information is needed for permitting. It is obtained by the Environmental staff, either through a hand held GPS unit, or through flagging and later collection by traditional survey (preferred method).
- o *Vertical control:* When a known datum is within a mile of the project, a level loop is run, providing accurate NGVD information. In a remote area more than a mile from a known datum, GPS will be used, which can result in the absolute elevation being inaccurate by as much as 8 inches. An effort should be made to tie down flood elevations to known elevations. However, relative elevations will be reliable within the project limits.

For some projects, additional information should be collected. For example, on culvert rehabilitation projects, if the shape of the existing culvert must be verified, the interior of the pipe or pipe arch should be surveyed. Points at the

top, bottom and the quarter points of the culvert should be taken at roughly 10 foot intervals along the centerline of the culvert.

For culvert replacement or culvert rehabilitation projects with fish passage concerns, grade control structures may be needed to maintain fish passage. If so, survey will be needed along the centerline of the stream at least 40 feet downstream of the end of any scour hole. Survey should extend a minimum of 20 feet on both sides of the stream or up to an elevation roughly 1/3 the height of the culvert. Depth of water at the upstream and downstream end of the culvert is also obtained by taking shots of the water surface.

Some projects will also need stream cross sections to create a hydraulic model. Generally, an absolute minimum of four sections of the stream is needed. The stream/river sections should include the streambed under water and the entire stream bank.

For larger projects, other means of collecting data should be considered. Photogrammetry may save time when many data points will be required. Fathometry may be preferred for very deep rivers or tidal areas. These options may be discussed with the Survey Coordinator.

2.6 Utilities and Right-of-Way

It is important to involve utilities and right-of-way team members in the project from the beginning. Considering the impacts of the design throughout the process will best address utility relocation issues and property owner concerns as they arise. Refer to Section 4.10 for utility attachment restrictions.

2.7 Alignments

2.7.1 General Highway Design Guidelines

In general, the alignment of the road is chosen first, which then determines the alignment of the bridge. Hydraulic, environmental, and economic concerns may result in an exception.

The Designer should refer to the “MaineDOT Urban & Arterial Highway Design Guide” for uniform design practices of approaches for collector roads, and to the current edition of AASHTO A Policy on Geometric Design of Highways and Streets for arterials. For local roads, the “MaineDOT Urban & Arterial Highway Design Guide” should generally be used; however, a lesser standard may be acceptable, particularly with low current traffic volumes, limited potential for growth, and potential adverse impacts to property owners, the environment, and economics of the area.

When the approaches to a bridge must be on a curved horizontal alignment, the Designer should keep any superelevation transitions off the bridge, if at all possible. The geometry of a superelevation transition can create an undesirable level area on the bridge deck, resulting in poor drainage, and can increase the cost of structural steel due to the complicated geometry.

2.7.2 Bridge Guidelines

2.7.2.1 Horizontal Alignment

When possible, a bridge should be located on a tangent section, since curvature increases the cost of the superstructure and can result in an undesirable safety situation during inclement weather. The “Plan Development and Estimating Guide” has details showing general bridge layout on a tangent, curve, and partial curve, as well as layout of a buried structure.

2.7.2.2 Vertical Alignment

When possible, the vertical alignment should be such that low point of the sag vertical curve is not at the bridge. A minimum 1% grade should be maintained across a bridge in order to facilitate positive drainage. If a 1% grade creates undesirable environmental or right-of-way impacts, then the grade may be reduced to 0.5%.

2.7.3 Clearances

2.7.3.1 Railroad

For new structures over railroads, the minimum clearances are shown in Figure 2-1, and are subject to the approval of the utility. The typical section shown should be used as a guide only. All railroad sections must be coordinated with the railroad on a project-by-project basis for approval.

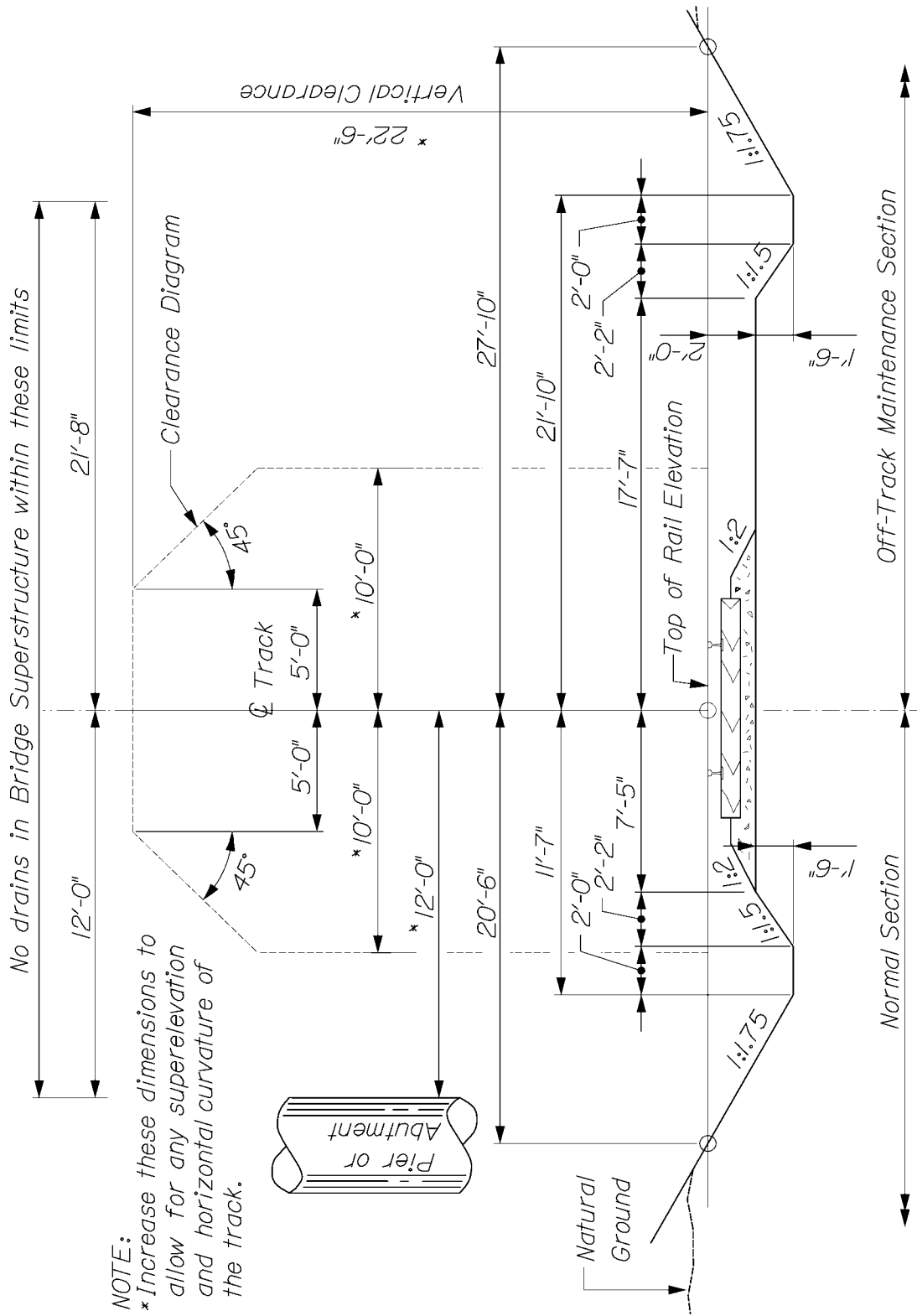


Figure 2-1 Typical Railroad Cut Section

The American Railroad Engineering and Maintenance of Way Association (AREMA) Manual for Railway Engineering (2002), Chapter 8 paragraph 2.1.5, requires that piers located within 25 feet of the tracks shall be of heavy construction or shall be protected by a crash wall.

2.7.3.2 Grade Separations

The legal minimum underclearance without posting is 14'-6".

For new structures over roads other than Interstate roads, the minimum underclearance is 15'-0" and the preferred underclearance is 15'-6". The minimum underclearance allows 6 inches for future pavement overlays and construction tolerances, and the preferred underclearance provides for an additional 6 inches of unknown conditions. The preferred underclearance is to be used for preliminary designs.

The corresponding underclearances for structures over the Interstate System are 16'-0" and 16'-6".

When a roadway is resurfaced under a structure, it may be necessary to excavate the existing pavement prior to placing new pavement in order to maintain the minimum underclearance and avoid the need for posting. In general, 16'-0" clearance for the Interstate and 15'-0" for other roads should be provided after resurfacing improvements are made, if other bridges on the corridor segment have corresponding minimum underclearances. To avoid posting, there should be an actual underclearance of 14'-6" minimum after improvements are made.

2.7.3.3 Underclearance for Stream Crossings

Refer to Section 2.3 Hydrology, Hydraulics, and Scour. For guidance on Coast Guard clearances and permits, refer to the Outside Agencies Chapter of the Bridge Program's "Project Management Guide."

2.7.3.4 Clearance Between Parallel Structures

In order to provide adequate room for certain maintenance activities such as painting and inspection, 10 feet minimum should be provided between parallel structures.

Under extreme circumstances, a 6 foot clearance may be allowed with concurrence from Bridge Maintenance.

2.7.3.5 Underclearances for Non-Vehicular Bridges

Non-vehicular bridges should meet the underclearance requirements in Sections 2.7.2.2 and 2.7.3.3.

2.8 Approaches

2.8.1 Roadway Widths

This section is a guide for use in determining the appropriate width of the approaches to a bridge. For geometric design criteria of bridge widths, refer to Section 4.1 Bridge Widths.

For projects on the NHS, widths must comply with the current edition of AASHTO A Policy on Geometric Design of Highways and Streets. Rural NHS roadways should not be designed for less than 40 mph. Refer to Figure 2-2 for the designated NHS in Maine.

All roads and streets (excluding the Interstate) are classified according to function. The proper function can be found in MaineDOT's ProjEx system for any given project. The functions are as follows:

- o Local roads
- o Minor and major collector roads
- o Minor and major arterials

Each of the classifications is further divided into two categories: urban and rural. For urban streets, existing approach widths should be investigated for their propensity to be widened or altered in the future. For rural roads, the Designer should determine from the Bureau of Planning whether the corridor is planned for widening in the future.

2.8.1.1 Local Roads

For local roads, the approach width should match the bridge width with the guardrail-to-guardrail width matching the rail-to-rail width on the bridge. Good engineering judgment is required when determining the appropriate width for a local road. Factors that need to be considered are:

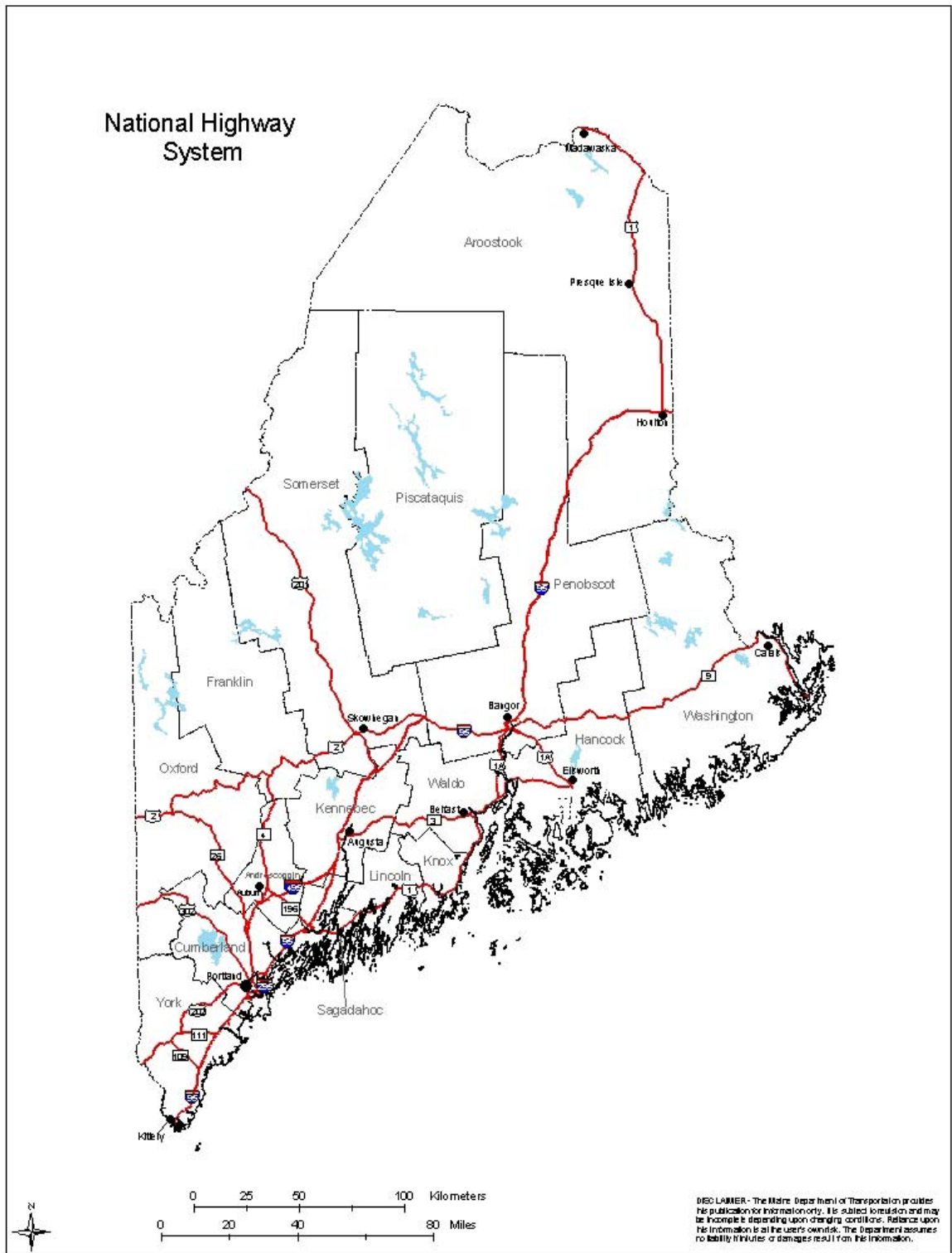


Figure 2-2 NHS in Maine

- Current right-of-way limits
- Geometric alignment
- Traffic volume
- Propensity for growth

2.8.1.2 Collector Roads

The approach guardrail (attached and immediate to the bridge) should be set at the same width as the bridge rail. For bridges on collector roads with extensive approaches, refer to the “MaineDOT Highway Design Guide” for appropriate shoulder widths and guardrail offsets.

2.8.1.3 Arterials

Roadway widths for approaches on arterials should comply with the latest AASHTO A Policy on Geometric Design of Highways and Streets.

2.8.2 Guardrail

2.8.2.1 General

On the NHS, terminal ends must meet the requirements of NCHRP 350 in conjunction with either guardrail type 3d on Interstate projects and 3c on non-Interstate NHS. Refer to Section 10 of the “MaineDOT Highway Design Guide” for further guidance. On non-NHS roadways with an AADT > 500, use a NCHRP 350 compliant system for an end treatment with guardrail type 3c as appropriate. On non-NHS roadways with AADT of 500 or less, use the Low Volume Guardrail End with guardrail type 3c as appropriate. For more information on guardrail types, refer to the Standard Specifications and Standard Details.

2.8.2.2 Guardrail Treatment on Local Roads

Bridge approach guardrails protect motorists from roadside hazards such as non-negotiable foreslopes, telephone poles, trees, streams, and rivers, and provide safe transitions to the bridge rail system. For guidance on bridge rail systems, refer to Section 4.4 Bridge Rail. Termination of these systems is controlled by the steepness of the foreslopes, location of obstacles, and the geometry of the stream crossings. Termination design criteria are presented in the current edition of the AASHTO Roadside Design Guide and the “MaineDOT Highway Design Guide”. The use of

these criteria can result in lengthy terminations and can extend projects beyond the lengths required to meet the objective of the project.

Bridge projects on local roads are intended to upgrade deficient structures and provide cost effective guardrail systems. This section provides design criteria for local bridge projects that minimize guardrail termination lengths and also eliminate the use of NCHRP 350 compliant end treatments in some instances.

The termination and NCHRP 350 compliant end treatment design criteria set forth in this section are intended for use only on roads for which the functional classification is local. Other projects should be designed in accordance with the guidelines and policy set forth in the “MaineDOT Highway Design Guide”.

Use the following definitions in this section:

- *Clear zone*: The clear zone is an unencumbered area measured perpendicular to the roadway that allows out of control vehicles leaving the roadway to recover.
- *Non-recoverable slope*: A slope that motorists can traverse but from which most motorists will be unable to stop or return to the roadway. Slopes that are between 4:1 and 3:1 are considered traversable but non-recoverable.
- *Critical slope*: A slope on which a vehicle is likely to overturn. Slopes that are steeper than 3:1 are considered critical.
- *Recovery area*: Sum of the clear zone and the non-recoverable and critical slopes.
- *Lateral extent of hazard*:

Stream that extends beyond the clear zone: The point where the outer limit of the recovery area intersects with the top of the non-negotiable slope at or near the stream edge.

Fixed object such as a tree, pole, etc.: The distance from the edge of the traveled lane to the far side of the hazard.

- *Runout path*: Theoretical path an out of control vehicle will follow as it leaves the roadway at the point of need.
- *Point of need*: The last point at the face of guardrail where a vehicle can leave the road and follow the runout path without traversing a critical slope or hitting a Deadly Fixed Object.

Refer to Figure 2-3 and Figure 2-4 for further guidance.

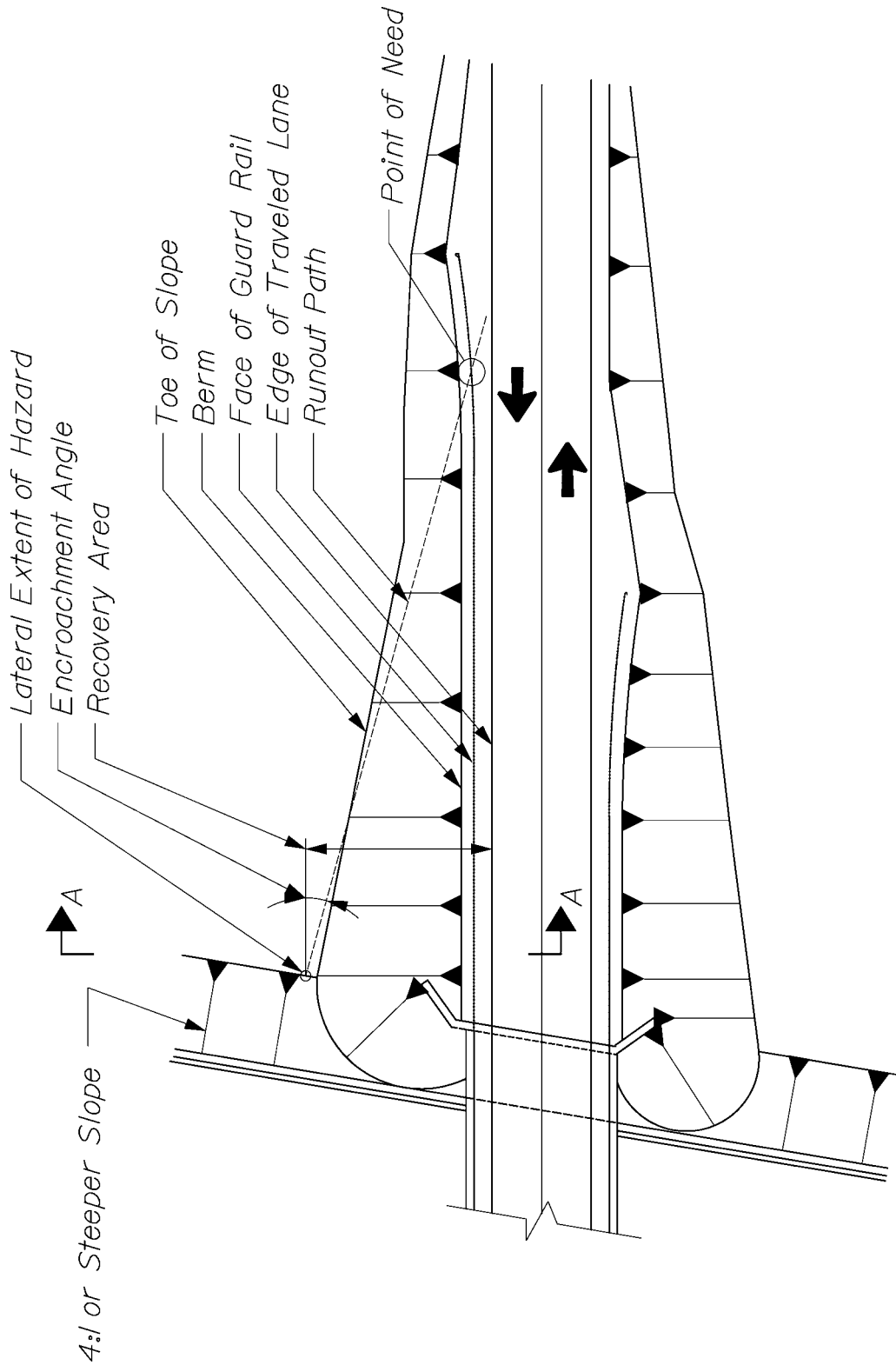


Figure 2-3 Point of Need Definition

Figure 2-3 Point of Need Definition

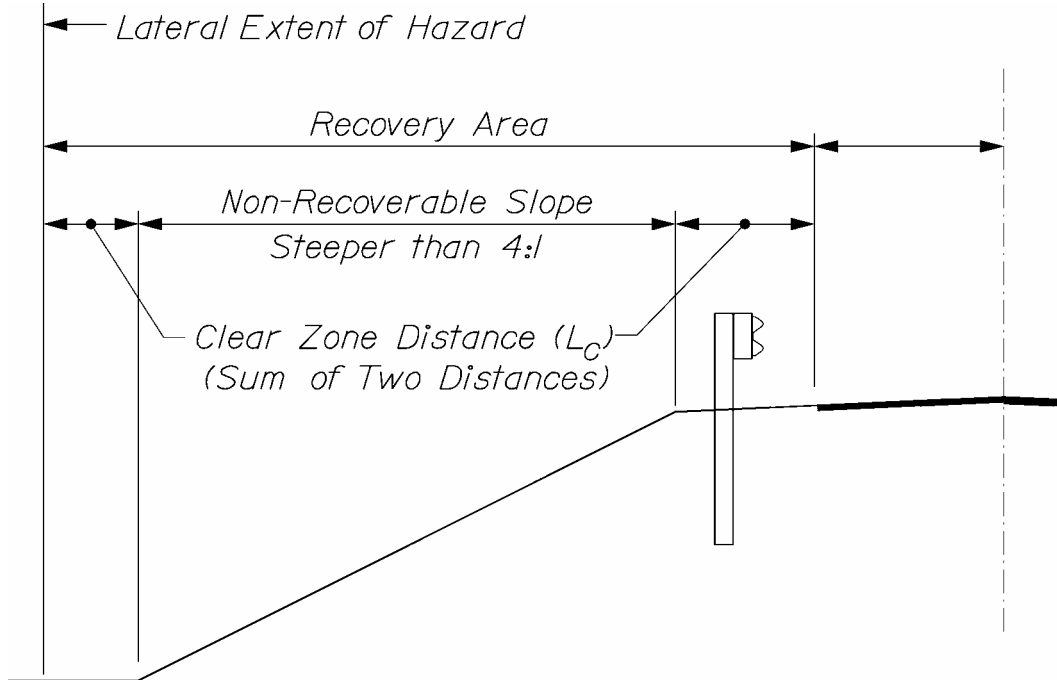


Figure 2-4 Lateral Extent of Hazard Definition

Procedure 2-1 has been developed to determine the proper treatment of the terminal end for the Leading End and Trailing End.

Procedure 2-1 Guardrail End Treatment on Local Roads

For the Leading End, follow the procedure below.

- a. Establish the clear zone distance (L_c) based upon the design future traffic volume and the design speed. (Refer to Table 2-4)
- b. Locate the lateral extent of hazard.
- c. Establish the runout path and the point of need by extending a line from the limit of hazard point to the face of guardrail at the encroachment angle based upon the design speed. (Refer to Table 2-5)
- d. Provide an end treatment beyond the point of need:

AADT > 500: Use an NCHRP 350 compliant end treatment system

AADT ≤ 500: Use a low volume guardrail end.

The use of NCHRP 350 compliant end treatment systems should be examined on local road projects where maintenance will be provided by the local government. These facilities may not be maintained, and after an NCHRP 350 compliant end treatment system is hit and damaged, it may be more dangerous than a standard flared terminal end.

CHAPTER 2 – PRELIMINARY DESIGN

- e. Where possible provide a minimum length of 100 feet from the bridge to the end of the guardrail. The length of the project should be extended if necessary to provide this minimum length of guardrail.

A minimum length of guardrail should be provided regardless of the project length to provide adequate protection at the approach rail - bridge rail interface.

Guardrail may be extended onto the approach transitions or even beyond the transitions by rehabilitating the existing shoulders and defining a limit of work beyond the end of the transition.

Table 2-4 Clear Zone

AADT (Future)	Clear Zone (Lc, ft)		
	30 mph	40 mph	50 mph
<200	5	7	8
200 to <400	6	8	10
400 to <800	7	10	12
800 to <2000	10	12	14
2000 to <6000	12	15	18
6000+	14	17	20

Table 2-5 Encroachment Angle

Design Speed	Encroachment Angle
30 mph	15°
40 mph	12°
50+ mph	10°

For the Trailing End, follow the procedure below.

- a. The required clear zone width for the trailing end (measured from the centerline of the road to the lateral extent of the hazard) is within the width of the adjacent lane plus the shoulder for an AADT less than 6000. Stream protection need not be considered unless the AADT equals or exceeds 6000, or unless terrain features (such as a stream which is skewed to or nearly parallel with the roadway) require consideration.
- b. Establish the point of need at the face of guardrail adjacent to the first 3:1 slope. (Where the transition from a 3:1 to a 2:1 slope begins.)
- c. Provide an end treatment beyond the point of need:
 - *AADT > 500: Use an NCHRP 350 compliant end treatment system.*
 - *AADT ≤ 500: Use a low volume guardrail end.*
- d. Where possible, provide a minimum length of 50 feet from the bridge to the end of the guardrail.

Other special conditions may also require consideration for guardrail treatment on local roads, including terrain features, approach curves, ditches, intersections, and driveways.

Certain terrain features can reduce the need for long guardrail lengths. If the calculated guardrail length exceeds the minimum requirement of 100 feet, examine the terrain along the runout path and within the clear zone. Will a motorist likely avoid the hazard by entering a field or open space before reaching the hazard? Will a motorist likely become hung-up in the brush before reaching the hazard? Is the stream bank flat (3:1 or flatter) and the stream shallow (3 feet or less at normal water) so that the motorist will be safer entering the stream than hitting the guardrail? These features must be evaluated on a project-by-project basis, and proposed guardrail reductions approved by the project team.

Longer guardrail lengths may be required to protect vehicles from utility poles and non-breakaway signs located within the clear zone.

When an approach curve is present, along with a high accident history, increasing the clear zone width, L_c , may reduce accident potential. For sharp approach curves, the runout path should follow a line tangent to the curve to the lateral extent of hazard.

Ditches may affect guardrail length. Trapezoidal approach ditch sections (2 feet wide at the bottom) should have 3:1 or 4:1 (preferred) foreslopes and 2:1 backslopes in areas where the ditches are parallel to the direction of travel. In areas where traffic could be expected to cross the ditch at a sharp angle such as the outside of a curve, the slopes should be flattened to conform to the recommendations in the AASHTO Roadside Design Guide.

If intersections, drives, or field entrances are found within the runout length, adequate sight distance must be provided. Guardrail should be wrapped into the entrance and terminated with a standard terminal end. NCHRP 350 compliant end treatment systems should be used on side roads where AADT exceeds 500.

The following Example 2-5 illustrates concepts shown in Procedure 2-1.

Example 2-5 Guardrail End Treatment on Local Roads

Given: Design Speed= 45 mph
AADT= 650
11 ft Lane width
4 ft to face of rail
3 ft from face of rail to berm

Problem: Determine the point of need for the leading and trailing ends.

Solution: Follow the Guardrail Treatment on Local Roads Criteria. Refer to Figure 2-5 and Figure 2-6.

Leading End

Step 1: Determine the clear zone distance from Table 2-4. The 45 mph design speed must be rounded to the next highest design speed given in the table, 50 mph. $L_c = 12$ ft

Step 2: Determine the lateral extent of hazard. In this example, the stream is the hazard. Since the stream extends beyond the recovery area, the lateral extent of hazard is the point where the limit of the recovery area meets the first non-recoverable slope (steeper than 4:1) at the edge of the stream.

Step 3: Establish the runout path. For the 45 mph design speed, round to 50 mph then select the encroachment angle from Table 2-5. Encroachment angle is 10°

Step 4: Locate the point of need. Extend the runout path to the face of guardrail. The intersection is the point of need. The length of guardrail exceeds the minimum of 100 ft.

Step 5: Provide an end treatment. The AADT exceeds 500, therefore use an NCHRP 350 compliant end treatment system. The last 3:1 foreslope should be located 50 ft from the point of need. The slope should be transitioned to 2:1 in 50 ft.

Trailing End

Step 1: From above, the required clear zone is 12 ft. Since the distance from the edge of the traveled lane (in this case the centerline of the roadway) to the face of rail of 15 ft is greater than the clear zone, stream protection is not necessary.

Step 2: Establish the point of need as the last 3:1 slope. In this case the side slope 50 ft from the bridge is 3:1, therefore use 50 ft from the bridge to the point of need.

Step 3: Since the AADT of 650 is more than 500, use an NCHRP 350 compliant end treatment system.

Step 4: The length of rail is 100 ft, exceeding the 50 ft minimum distance from the bridge.

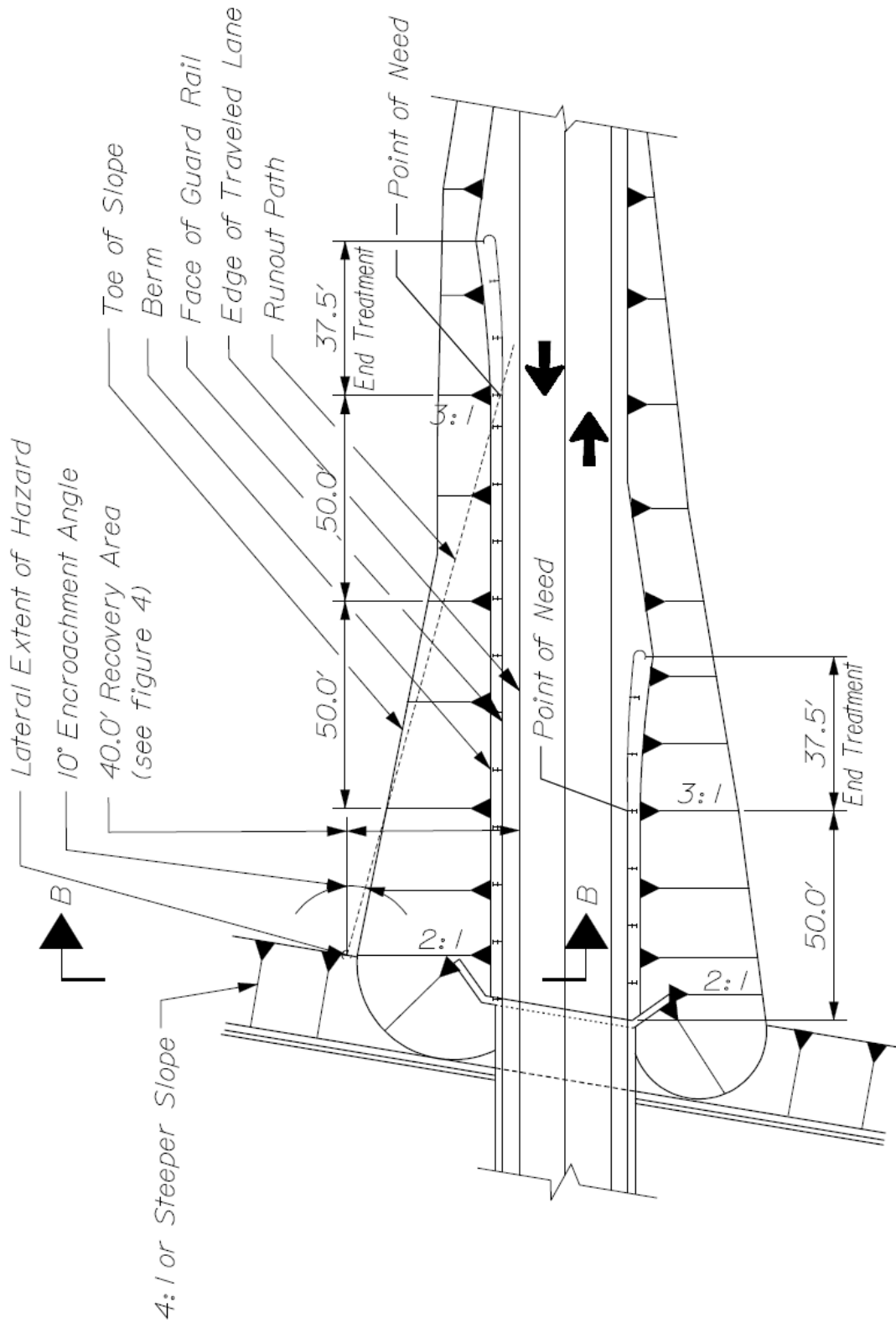


Figure 2-5 Point of Need Example

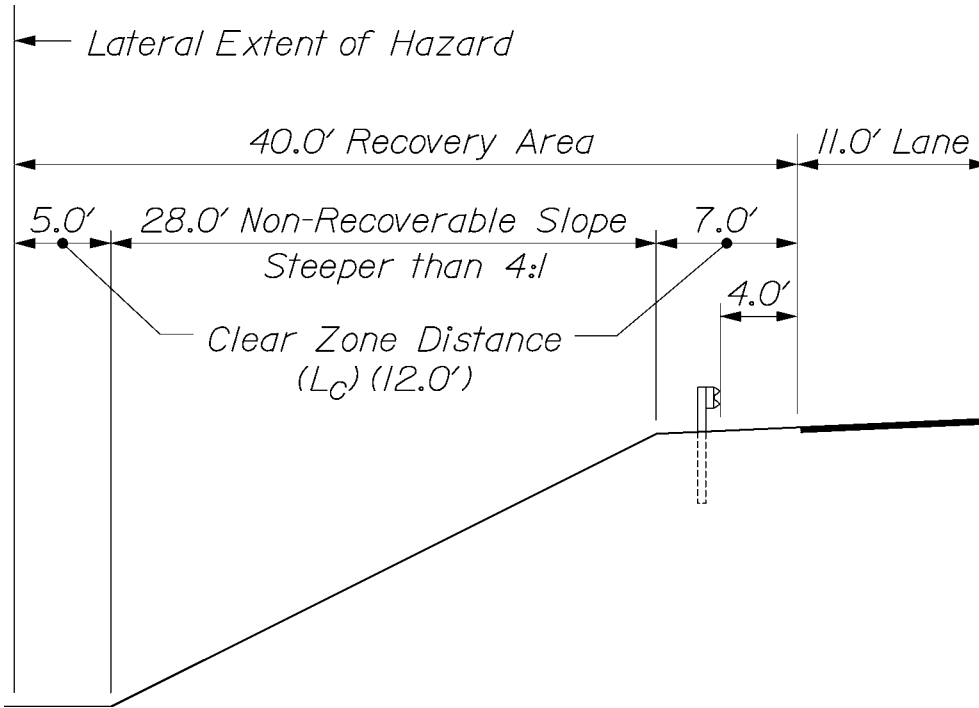
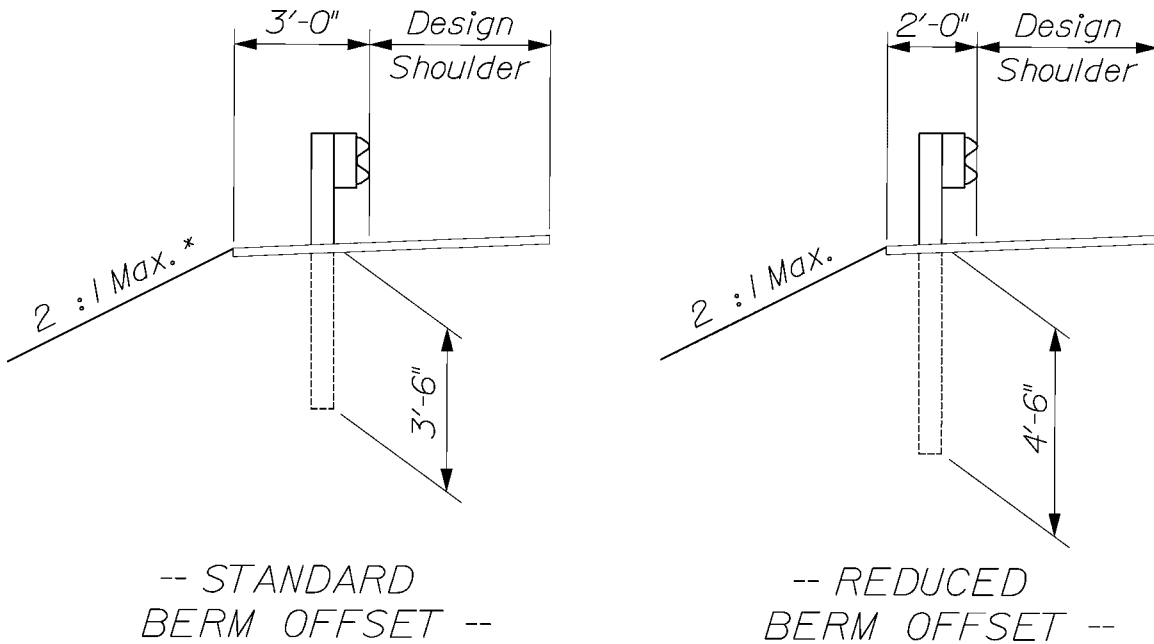


Figure 2-6 Lateral Extent of Hazard Example

2.8.3 Reduced Berm Offset

For projects on low volume, low speed local roads, consideration may be given to reducing the 3 foot offset from the face-of-guardrail to the berm to 2 feet in order to minimize wetland, right-of-way, or other construction impacts.

When reduced berm offsets are utilized, the guardrail posts must be lengthened and the embedment increased, as shown in Figure 2-7.



* Unless stabilized with riprap or other acceptable method

Figure 2-7 Reduced Berm Offset

2.8.4 Pavement Design

2.8.4.1 General

A. Layer Thickness

Use Table 2-6 for maximum and minimum layer thickness for a particular grade of pavement, in order to achieve the required density. Each grade may require more than one layer.

Table 2-6 Pavement Layer Thickness

Item Number	Description	Minimum Thickness (in)	Maximum Thickness (in)
403.210	3/8 in	1	1-1/2
403.208	1/2 in	1-1/8	2
403.207	3/4 in	2	3-1/4
403.206	1 in	2-1/2	4

B. Layers Across Roadway

Use Table 2-7 to estimate typical pavement layer thickness for traveled way and shoulders on projects that have bridges with wearing surfaces. For wearing surfaces on bridges, refer to Section 4.6 Wearing Surfaces. For approaches over buried structures, refer to the Highway Design Guide for guidance on layer thicknesses.

Table 2-7 Number of Layers Across Roadway

Travelway		Shoulder		Mix Type
Depth (inches)	Number of Layers	Min. Depth (inches)	Number of Layers	
3	2	3	2	9.5 mm or 12.5mm
4	1 @ 1-1/2" 1 @ 2-1/2"	3	2	9.5 mm or 12.5mm 12.5 mm or 19.0 mm
5	2 @ 1-1/2" 1 @ 2"	3	2 1	9.5 mm or 12.5mm 12.5 mm or 19.0 mm
6 or greater	2 @ 1-1/2" X @ X" *	3	2 1	9.5 mm or 12.5mm 12.5 mm or 19.0 mm

* Add and adjust layer thicknesses as needed.

C. Shoulder Pavement

If the proposed shoulder widths are 4 feet or less and the travelway pavement depth is 5 inches or less, the travelway pavement depth should be carried through the shoulders at the same slope as the travelway.

For bridge projects with short approaches where the proposed shoulder widths are 5 feet or greater and the travelway pavement is 5 inches or less, the travelway pavement depth may be carried through the shoulders.

For longer approaches with heavy traffic, intersections, and/or sharp horizontal curves with off-tracking concerns, the Designer should refer to the Highway Design Guide for guidance on approach shoulder width and thickness.

For further guidance, the Designer should consult with a pavement technical resource person. Paving the shoulder at the same depth as the travelway allows the shoulders to be paved along with the mainline. This shortens construction time and eliminates the need to place shoulder material and regrade the shoulders, resulting in cost savings that will more than offset the cost of the extra pavement.

2.8.4.2 Arterials and Collectors

Pavement for approaches located on all arterials and collectors, on and off the NHS, should be designed in accordance with the 1993 AASHTO design standards. The DARWin™ Pavement Design System designs pavement and aggregate subbase course gravel thicknesses based on the 1993 AASHTO Standards. Establishment of a new design standard is currently in progress.

For assistance in determining the Terminal Serviceability and Reliability Level (%), consult with a Designer in the Urban and Arterial Program.

Table 2-8 contains sample input data for the DARWin™ program.

Table 2-8 DARWin Input

18-kip ESALs over Initial Performance Period	Use equivalent 18k loads from AADT, expanded over the entire pavement design period, typically 20 years. Example: 95 axles/day x 365 days/yr. X 20 yr. = 693,500 ESALs	
Initial Serviceability	4.5	
Terminal Serviceability	2 on Local Roads 2.5 on Arterials and Collectors	
Reliability Level %	95% on Interstate 95% on NHS 90%-95% on Rural State Routes: look at traffic volumes 85%-95% on Urban State Routes: Look at traffic volumes, turning movements, braking movements. 85% on Local Roads	
Overall Standard Deviation	0.45	
Roadbed soil Resilient Modulus	Given Soil Support	M _r (psi)
	3.0	2800
	3.5	3600
	4.0	4300
	4.5	5100
5.0	6100	
Staged Construction	1	
Layer Coefficients	Top 4 inches (maximum) of pavement	0.44
	Pavement below top 4 inches	0.34
	Aggregate Sub base Course Gravel	0.09
	Reclaim	0.14
	Reclaim with additive	0.22

2.8.4.3 Local Roads

Pavement on approaches located on local roads can be designed using Table 2-9.

Table 2-9 Pavement & Subbase Thickness

Equivalent Daily 18 k Single Axle Application	Pavement Thickness (in)	Aggregate Subbase Course - Gravel (in)	Total Subbase and Pavement Thickness (in)
0-25	3	21	24
26-50	3	24	27
51-100	3	27	30
101-150	4	26	30
>150		Design According to Collector and Arterial Standards, using Terminal Serviceability = 2	

2.8.5 Approach Drainage

Well-drained pavements can outlast poorly drained ones by at least three to four times. When most subgrade soils are compacted sufficiently to support vehicle loads, their permeabilities are cut down to a level that allows only miniscule amounts of water to drain downward (Cedergren, 1989). Positive drainage of the pavement (through crowning) and subgrade is critical to the long-term performance of the roadbed. Total drainage design should be studied, with reference to the drainage section of the “MaineDOT Urban & Arterial Highway Design Guide”.

In planning approach construction, the subgrade layer should be allowed to daylight on the foreslope of the roadway a minimum of 12 inches above the ditch line. If it is not possible to daylight the subgrade soils in this manner, consideration should be given to the use of an underdrain. Where underdrain is used, it must be positively drained away from the roadway.

Water should not be allowed to drain into the underside of slope protection. Permanent erosion control measures should be considered at the bottom of ditches.

2.8.6 General or Local Conditions

Good engineering judgment is required in all locations to determine the overall needs of the community by taking into consideration safety, future growth, and current needs. The Designer should also consider the geometric configuration of the corridor adjacent to the project during the design process. The design should reflect aesthetic, scenic, historic, and cultural considerations.

2.9 Structure Type Selection

A multitude of issues must be considered when the Structural Designer chooses the best structure type for a given project. The project team will contribute input according to each member's expertise.

2.9.1 *Span*

Span length will influence the optimal structure type and section to use. Spans less than 50 feet are discussed in Section 1.5 Small Bridge Initiative. Longer spans will generally be girder/deck bridges made of either steel or concrete. Rolled steel beams and precast, prestressed concrete box beams are used up to about 100 feet. Precast, prestressed concrete girders are used up to about 150 feet. Welded steel girders are used up to about 250 feet due to the practical limit of about 150 feet for shipping pieces. Longer spans will require steel girders with additional field splices, steel box girders, or segmental concrete girders.

The optimal span configuration will depend upon the cost of the proposed substructure units. Fewer piers will reduce the overall substructure cost, but will increase the span lengths and overall superstructure cost. Often the Structural Designer must balance the cost of the superstructure with the cost of the substructure to determine the best design.

2.9.2 *Maintenance of Traffic*

If staged construction is planned, the Structural Designer must lay out the proposed traffic scheme to be certain the existing and proposed bridges can support the traffic. The configuration of the existing bridge girders must be examined to determine the width remaining to support traffic once some of the girders are removed. Precast deck panels may be preferred for staged construction projects due to faster construction times. On precast structures, the width of available precast units must be considered.

The ability of the proposed structure to support traffic before the structure is complete must also be explored. For example, a structural plate structure is very difficult to stage, due to difficulty connecting the plates in place, the need to temporarily reinforce the ends, and concerns about non-uniform backfill.

2.9.3 *Constructability*

The Structural Designer and Construction Resident must agree that the proposed structure can be constructed. This can be of particular concern on rehabilitation or staged construction projects. The sequence of construction and an acceptable method of construction of both the foundation and structure must be studied before submitting a considered design. In particular,

adequate space must be available for the Contractor to perform the necessary work, and existing subsurface and stream conditions must be carefully examined. Difficulty in construction of substructure units due to site conditions may favor the use of longer more expensive superstructure units. Other examples of common constructability issues include the method of cofferdam construction, the use of mechanical couplers in tight spaces, and the limitation of commonly used forms in the construction of a wide slab overhang.

As one form of scour protection, consideration should be given to the practice of leaving the sheet piling used for cofferdams in place and cutting them off at the streambed elevation after construction is complete. Refer to Section 5.2.3 Cofferdams.

2.9.4 Environmental Impact

The goal when applying for environmental approvals is always to avoid or minimize environmental impacts. The Structural Designer often must balance this reduction of impacts with the additional cost that may be added to the project. With this in mind, the Structural Designer will design a water crossing bridge long enough to minimize stream impacts. In some cases, tight in-stream work windows may force the design to stay out of the stream altogether. Return wingwalls and headwalls on culverts are used to minimize impacts to the stream and to adjacent wetlands. Reduced berm offsets are considered on local roads to keep toes of slope out of wetlands.

For culvert-type structures, attention must be given to the impact of the structure bottom on the stream. In some cases, environmental restrictions may force the Structural Designer to use a three-sided structure without a bottom instead.

2.9.5 Right-of-Way Impact

Whenever possible, the impacts to adjacent property owners should be kept to a minimum. Methods such as wingwalls and reduced berm offsets on local roads can be used. Other considerations include maintaining accessibility to homes and businesses during and after construction.

The cost of right-of-way issues can impact both the budget and schedule. The lengthy right-of-way process can cause project delays when people are displaced from acquired buildings. Dollar cost of property acquisition can also be high in some areas. The existence of gas stations, mills, or factories can herald the presence of hazardous materials that must be removed at significant cost.

2.9.6 Maintainability

Long-term maintenance is always part of the equation when determining the optimal structure type, and has influenced Bridge Program policy throughout this guide. For example, weathering steel has lower maintenance cost than painted steel. Policies have been developed for issues such as these by balancing first cost with maintenance cost. The Designer should always try to keep future maintenance costs as low as practical.

The Designer should keep the following in mind when choosing design options:

- o Look at how the bridge will be maintained. Will high traffic volumes limit maintenance activities? Will maintenance be very expensive? If so, it will be even more important to design low frequency maintenance structures.
- o Consider how parts of the bridge will be repaired, such as bearings. Is there room for temporary support? Is there adequate access? Catwalks should be considered around abutments and piers for large or extraordinary projects. The bearing seats for abutments and piers should be wide enough to accommodate jacks for future bearing replacements.
- o Is the bridge wide enough to maintain traffic during deck repairs and wearing surface replacements? Is the approach wide enough where return wingwalls are used?
- o Use standard sizes and coatings when possible to facilitate prompt repair with off-the-shelf items.
- o Consider the need to remove winter sand from bridge seats and rails. Avoid designs that allow winter sand accumulation on bearings and beam ends.
- o Consider under-bridge crane limitations for inspection. Vertical reach will limit fences to 6 feet high, and horizontal reach will limit sidewalks to 8 feet wide.
- o Consider bridge width needs for snowplows to facilitate plowing and to limit potential damage in accordance with Section 4.1 Bridge Widths.
- o Consider the need to inspect substructures for scour. If inspection is impossible due to high velocities, provide additional protection.

2.9.7 Historical/Archeological Issues

It is critical that any project that has historical or archeological interest is flagged early in the process. Working with the Maine Historic Preservation Commission (MHPC) and relevant historic districts as the design is developed will save considerable time in the process.

2.9.8 Cost

The Structural Designer should attempt to find the lowest cost option that satisfies the requirements of the applicable code, MaineDOT guidelines, and the traveling public, but does not sacrifice quality. First cost must be considered, as well as life cycle cost in some cases (refer to Section 2.2 Economic Comparisons). The program cost should be identified, and every attempt made to design a project that falls within that budget.

2.9.9 Aesthetics

The consideration of aesthetics in every design is encouraged. Often there are low cost methods that can be incorporated into a design that can greatly increase the aesthetic value of the project. Refer to Section 1.7 Aesthetics for more discussion.

2.10 Subsurface Exploration Programs

2.10.1 Boring Program

The boring program and geotechnical investigation is developed by the Geotechnical Designer. The boring program includes a summary of all geotechnical activities such as the boring identification number, boring station and offset (if available), boring termination requirements, sampling requirements, and in situ testing requirements. It also includes a plan showing the proposed boring locations for the project containing the following information:

- Title block
- 1:25 plan view of the existing structure and the proposed structure (if known)
- Proposed boring locations indicated by the standard symbol

Each boring location must have a unique boring identification number in the following format: XX-YYYY-ZZZ. The X terms in the boring number will be “BB” for bridge borings. The Y terms will be the first letter of the town(s) and

the initials of the crossing. The Z terms will be a 100, 200, 300, etc. series number. For example, the first series of borings for a bridge crossing Noname River in Anytown is designated as BB-ANR-101, BB-ANR-102, etc. Any additional borings conducted at a site after the initial borings have been completed will be designated as 200 series, 300 series and so forth.

One copy of the boring program is submitted to the Structural Designer. One copy will remain in the Bridge Geotechnical File. Two copies will be sent to MaineDOT Materials, Testing, and Exploration Division (MTED) in Bangor: one for the MTED file and the other for use in the field.

2.10.2 Exploration Program Objectives

For traditional bridge structures, the cost of a boring is small in relation to the overall foundation cost. The knowledge gained from borings permits the use of appropriate design techniques and allows for less conservative designs. Without adequate boring data, evaluating geotechnical alternatives becomes more difficult, and the Geotechnical Designer must rely on more conservative designs.

Planning a boring program should include:

- Determining the depth and location of borings, test pits, and/or auger probes, as necessary
- Establishing the methods of soil sampling and testing

The number, depth, spacing, and character of tests to be made in any individual boring program are dependent upon site conditions, type of structure, the structure's performance, and design requirements. Due to the site specific nature of each subsurface exploration, there is no preferred approach for establishing the program. Certain general principles guiding the development of a subsurface investigation, such as soil sampling and in-situ testing, are a necessary part of every investigation.

The boring program is established by the Geotechnical Designer and reviewed with the Structural Designer. A boring program based on these guidelines will produce the minimum geotechnical information to evaluate a typical bridge structure site.

2.10.3 Preliminary and Final Borings

A subsurface investigation may be required during the preliminary stages of a project at a time when the alignment, the location of abutments or the location and number of piers are not yet established. In these instances, preliminary borings may be conducted to yield only sufficient soil information to enable the Structural Designer to:

- Provide subsurface information for development of foundation alternatives for the PDR
- Establish the vertical and horizontal roadway alignment alternatives
- Locate proposed substructure units
- Prepare the preliminary cost estimates

At a minimum, the preliminary boring program shall include:

- One boring per substructure unit
- Consideration of a 10 foot deep rock core should be made at this time if the information would influence the foundation design

Preliminary borings should not be used for final design purposes. A final boring program should be developed and carried out in accordance with the frequency and depth requirements in Table 2-10, Table 2-11, and this Section.

If the project alignment, location of abutments, and location and number of piers is established with certainty during preliminary design stage a preliminary boring program should not be conducted.

2.10.4 Number and Layout of Borings

Borings shall be taken for every:

- bridge
- retaining wall
- metal pipe, plate arch, pipe arch, or box, with a span greater than 8 ft
- concrete arch or box with a span greater than 8 ft
- high-mast light foundation
- single support cantilever sign foundation
- other traffic or sign supports which require a foundation.

The borings should be performed using cased, wash boring techniques. In some instances, open-hole hollow stem auger and/or solid stem auger drilling methods may be used.

For final design, the number and layout of borings should be determined as suggested in Table 2-10. The guidelines shown in Table 2-10 are the minimum requirements to evaluate a site for design. The number and layout

of borings; however, will also depend on the phase of the investigation being performed. For feasibility studies or preliminary PDR investigations, fewer borings and very wide boring spacings may be acceptable, especially if the subsurface conditions are uniform.

The location and frequency of drilling and sampling depend on the type and critical nature of the proposed structure, the soil and bedrock characteristics, the variability in subsurface conditions, the loads to be imposed on the foundations soils, and the structures performance and design requirements.

Table 2-10 is intended to be used as a guideline. Actual determination of number and layout of borings is at the Geotechnical Designer’s discretion.

Table 2-10 Guidelines for Boring Number and Layout

Foundation Type/ Geotechnical Feature	Boring Layout
Bridge Foundations	<p>For piers and abutments less than 100 feet in length, provide a minimum of one boring at each pier and abutment. Boring locations should be staggered diagonally at opposite ends of adjacent footings.</p> <p>For piers and abutments over 100 feet in length, provide a minimum of two borings. Borings should be located at the extreme corners of each substructure.</p> <p>For spread footings on sloping bedrock surfaces, additional borings or probes may be required.</p> <p>Additional borings should be provided in areas of erratic subsurface conditions.</p>
Retaining Walls	<p>For retaining walls up to 100 feet in length, provide a minimum of one boring at the wall end. For abutment wingwalls which measure less than 30 feet in length, the abutment borings may suffice.</p> <p>For retaining walls more that 100 feet in length, the spacing between borings should be no greater than 100 feet.</p> <p>For walls ≤ 20 feet high, use a maximum boring spacing of 100 feet. For walls ≥ 20 feet high use maximum boring spacing of 50 feet, regardless of the wall length.</p>

	<p>Additional borings inboard and outboard of the wall line to define conditions at the toe of the wall and in the zone behind the wall to estimate lateral loads and anchorage capacity may also be required.</p>
Roadways	<p>The spacing of borings along the roadway alignment generally should not exceed 200 feet.</p> <p>The spacing and location of the borings should be selected considering the geologic make up of the soil and bedrock continuity within the project area. The objective is to define the vertical and horizontal boundaries of distinct soil and rock units within the project limits.</p>
<p>Metal Pipes Metal Arches Metal Box Culverts Concrete Box Culverts</p>	<p>Provide a minimum of one boring at each buried metal pipe, arch, box culvert, and concrete box culvert, with a span greater than 8 feet.</p> <p>Additional borings should be provided for long culverts or in areas of suspected erratic subsurface conditions.</p> <p>For culverts up to 50 feet in length, two borings are required. For culverts longer than 50 feet, three borings are required.</p>
Cuts	<p>A minimum of one boring should be performed for each cut slope less than 100 feet in length.</p> <p>For cuts more than 100 feet in length, the spacing between borings along the length of the cut should generally be between 100 and 200 feet.</p> <p>At critical locations and high cuts, provide a minimum of three borings in the transverse direction to define the existing geological conditions for slope stability analyses. In an active slide area, place at least one boring upslope of the sliding area.</p>
Embankments	<p>A minimum of one boring per 100 feet should be performed for each embankment.</p> <p>For embankments more than 100 feet in length, the spacing between borings along the length of the embankment should generally be between 100 and 200 feet.</p> <p>At critical locations and high embankments, provide a minimum of three borings in the transverse direction to</p>

	define the existing geological conditions for slope stability analyses. In an active slide area, place at least one boring upslope of the sliding area.
Staged Construction	Provide at least one boring to determine the constructability and design of staged construction sheet piling.

2.10.5 Depth of Borings

Borings shall be advanced to a depth which will provide sufficient information to evaluate bearing capacity, settlement, slope stability, pile capacity, and other geotechnical design factors as required by the site conditions. Table 2-11 provides guidelines for selecting minimum boring depths. Ultimately, the final boring depth at a specific site is at the discretion of the Geotechnical Designer. Field judgment exercised during the investigation phase will further define the boring program and ultimately satisfy the investigation requirements necessary for design of the foundation system.

Frequently, it may be necessary to extend borings beyond the minimum depths shown in Table 2-11. Deeper borings can allow for better definition of the geologic setting at the site, assist in determining the depth and engineering characteristics of soft soil, and assure that sufficient information is obtained when the structure requirements are not clearly defined at the time of drilling. Where borings are drilled to bedrock, it is recommended that a minimum 10 foot length of bedrock core be obtained to verify that the borings has not terminated on or within a boulder.

Subsurface investigation programs must be flexible to adjust to variations in subsurface conditions encountered during drilling. On critical projects, the Geotechnical Designer should be present during the field investigation in order to make field decisions regarding boring depth, in-situ testing and additional borings. Open lines of communication with the Structural Designer to discuss unusual field findings and changes are necessary during the investigation program.

Table 2-11 Minimum Requirements and Guidelines for Boring Depths

Areas of Investigation	Boring Depth
Bridge Foundations - Spread footings	No boring shall be less than 10 feet below the preliminary bottom of footing elevation. For abutments on slopes, borings shall extend at least 20 feet below the proposed bottom of footing elevation.

	<p>For isolated footings of length L and width B, where $L \leq 2B$, borings shall extend a minimum of two footing widths (2B) below the footing bearing level.</p> <p>For isolated footings where $L \geq 5B$, borings shall extend a minimum of five footing widths (5B) below the footing bearing level.</p> <p>For $2B \leq L \leq 5B$, minimum boring depths shall be determined by linear interpolation between a depth of 2B and 5B below the footing bearing level.</p> <p>For spread footings founded directly on bedrock, the length of the bedrock core should be no less than 10 feet.</p> <p>Selection of boring depths at river and stream locations must consider the potential scour depth of the stream bed.</p>
<p>Bridge Foundations - Deep Foundations</p>	<p>A minimum of one boring shall be made to bedrock under each substructure unit that is founded on piles or shafts.</p> <p>For piles or shafts bearing on soils, borings shall extend below the anticipated pile or shaft tip elevation by a minimum of 20 feet, or a minimum of 2 times the maximum pile group dimension, whichever is deeper.</p> <p>For piles bearing on bedrock, a minimum of 10 feet of bedrock core shall be obtained at each boring location to verify that the boring has not terminated on a boulder, and to determine RQD for a 10 feet bedrock core.</p> <p>For drilled shafts supported in or on bedrock, a minimum of 10 feet of bedrock core, or a length of bedrock core equal to at least 3 times the shaft diameter, shall be extended below the preliminary shaft tip elevation.</p>
<p>Retaining Walls</p>	<p>Borings shall have a minimum depth of 0.75 to 1.5 times the height of the wall below the anticipated bottom of footing. Where the soil type indicates possible deep stability or settlement problems, borings should extend to an underlying competent stratum.</p> <p>Boring depth for sheet piling should extend below the final ground line to a minimum of 2 times the exposed</p>

	<p>wall height.</p> <p>For wall supported on deep foundations, use the criteria above for deep bridge foundations.</p>
Roadways	Borings shall have a minimum depth of 5 feet below the proposed subgrade level.
Culverts, Boxes, Arches	Borings shall have a minimum depth equal to twice the backfill embankment height, unless a hard stratum is encountered above this depth. Where soft strata are encountered which may present stability or settlement concerns, the borings should extend to an underlying competent stratum.
Cuts	Borings shall extend a minimum of 15 feet below the anticipated depth of the cut at the ditch line. Boring depths should be increased where base stability is a concern due to soft soils, or in locations where the base of the cut is below groundwater level.
Embankments	Borings shall extend a minimum depth equal to twice the embankment height, unless a hard stratum is encountered above this depth. Where soft strata are encountered which may present stability or settlement concerns, the borings should extend to an underlying competent stratum.
Staged construction	The boring depth for sheet piling should extend a minimum of 2 times the exposed wall height.

2.10.6 Standards and Guidelines for Borings, Sampling and In-Situ Testing

Subsurface field exploration by borings should be conducted in accordance with applicable FHWA procedures, MaineDOT procedures, and the AASHTO/ASTM standards listed in Table 2-12. Standard procedure should always be followed as improvisation of investigative techniques may result in erroneous or misleading results which may have serious consequences on the interpretation of the field data. All sampling techniques and intervals shall be approved and/or determined by the Geotechnical Designer during drilling activities.

Borings for structure foundations and geotechnical features shall be cased wash borings having a minimum diameter of 3.0 inches. Standard penetration tests (SPT) should be performed for each boring at 5 foot intervals and at

changes in strata. Continuous sampling is recommended for the top 15 feet when footings are to be placed on native soils.

Soft ground conditions will require undisturbed sample explorations or in-situ testing. Undisturbed Shelby tube samples should be obtained at 5 foot intervals in at least one boring in cohesive soils. For cohesive soils greater than 30 feet in depth, tube samples interval can be increased to 10 feet. In silt-clay deposits, in situ vane shear strength tests are recommended at 5 to 10 foot intervals.

A minimum of 10 feet of bedrock shall be cored in borings reaching bedrock. The minimum diameter of bedrock core shall be 1.88 inches (NQ-size). The Rock Quality Designation (RQD) shall be calculated for all rock cores prior to transportation of the core.

Visual identification of the soil samples shall be performed by the field inspector in accordance with the Maine Department of Transportation “Key to Soil and Rock Descriptions and Terms”, April 2004.

All pertinent boring identification data, test data, visual classification of soil and rock, and changes in soil stratum shall be recorded in accordance with the “MaineDOT Soil/Rock Exploration Log”, and the MaineDOT “Visual Identification Rock Cores Log”.

Table 2-12 AASHTO, ASTM, and MaineDOT Standards for Field Investigations

Standard			Test / Practice
AASHTO	ASTM	MaineDOT	
	D 1452		Practice for Soil Investigation and Sampling by Auger Borings
T 206	D 1586		Method for Penetration Test and Split-Barrel Sampling of Soils
T 207	D 1587		Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
T 223	D 2573	MaineDOT “ <i>Vane Shear Testing Recommended Practice</i> ”, Feb. 2001	Test Method for Field Vane Shear Test in Cohesive Soil
	D 2113		Practice for Rock Core Drilling and Sampling for Site

			Investigation
	D 4220		Practice for Preserving and Transporting Soil Samples
	D 5079		Practice for Preserving and Transporting Rock Core Samples
		MaineDOT “ <i>Key to Soil and Rock Descriptions and Terms</i> ”, rev. April 2004	Practice for Visual Description of Soils and Rock
	D 6032		Method for Determining Rock Quality Designation of Rock Core
T 86	D 420		Standard Guide to Site Characterization for Engineering Design and Construction Purposes
		MaineDOT Soil/Rock Exploration Log	Practice for Visual Description of Soils and Rock
		MaineDOT Visual Identification Rock Cores Log	Practice for Visual Description of Rock Core

2.10.7 *Presentation of Subsurface Exploration Data*

2.10.7.1 Boring Log Sheets

Logs of all borings, test pits, and/or auger probes taken at a site must be transcribed to plan sheets. All borings conducted at the site shall be represented, including exploratory borings and those conducted for abandoned alignments. Laboratory test results should also appear on the boring logs.

The boring logs shall be drafted using the format of the MaineDOT LOGDRAFT Boring Log. LOGDRAFT supports output of AutoCAD DXF Files, which aids in the transcription of the boring logs to plan sheets with a title block called “Boring Logs”.

2.10.7.2 Boring Location Plans and Interpretive Subsurface Profiles

A longitudinal profile graphically depicting the subsurface conditions should be developed from all field explorations and lab tests. Approximate soil layer boundaries and accurate soil descriptions should be established for all

soil deposits. If the boring is terminated in bedrock, the approximate bedrock profile and accurate bedrock description should be established. Subsurface profiles should include the visual description of each soil deposit observed, bedrock description and profile, groundwater level, and special items such as boulders or artesian pressure, as applicable.

The subsurface profile can be presented with reasonable accuracy and confidence at the location of the borings. The Geotechnical Designer may present a continuous subsurface profile that shows an interpretation of the location, extent and nature of subsurface deposits between borings. Caution should be exercised in the presentation and interpretation of soil and geologic data between borings.

The location of the borings, augers, and/or test pits, as applicable, and the subsurface profile should be presented on sheets with a title block called "Boring Location Plan and Interpretive Subsurface Profile".

The Geotechnical Report should be accompanied by the following plan sheets:

- Boring Logs
- Boring Location Plan
- Interpretive Subsurface Profile

Where possible, these sheets may be combined to reduce the number of plan sheets.

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Chapter 3

LOADS



East, Center, and West Bridges, Fairfield-Benton



Morse Bridge, Rumford

CHAPTER 3 - LOADS

3	LOADS	3-1
3.1	General.....	3-1
3.2	MaineDOT Live Load Policy (New and Rehabilitation)	3-2
3.3	Thermal Effects	3-2
3.4	Construction Loads.....	3-3
3.5	Railroad Loads	3-3
3.6	Earth Loads	3-3
3.6.1	General.....	3-3
3.6.2	Presence of Water.....	3-4
3.6.3	Earthquake	3-4
3.6.4	Lateral Earth Pressure.....	3-4
3.6.5	Active Earth Pressure Coefficient.....	3-5
3.6.5.1	Coulomb Theory.....	3-5
3.6.5.2	Rankine Theory	3-7
3.6.6	Coulomb Passive Lateral Earth Pressure Coefficient.....	3-8
3.6.7	Lateral Earth Pressures for Unconventional Retaining Walls	3-9
3.6.7.1	Mechanically Stabilized Earth Walls.....	3-9
3.6.7.2	Prefabricated Modular Walls	3-9
3.6.8	Surcharge Loads – Live Load Surcharge	3-9
3.6.9	Passive Earth Pressure Loads	3-10
3.7	Seismic.....	3-10
3.7.1	General.....	3-10
3.7.1.1	Seismicity of Site	3-10
3.7.1.2	Geotechnical Characteristics of the Site.....	3-11
3.7.1.3	Functional Importance.....	3-11
3.7.1.4	Major or Minor Structures	3-11
3.7.1.5	Structure Type and Detail.....	3-11
3.7.2	Seismic Analysis.....	3-13
3.7.3	Substructure	3-14
3.7.4	Embankments & Embankments Supporting Substructure Units.....	3-15
3.7.4.1	Seismic Slope Stability	3-15
3.7.4.2	Liquefaction and Seismic Settlement	3-15
3.8	Non-Vehicular Bridges.....	3-16
3.9	Ice Loads	3-16
3.9.1	General.....	3-16
3.9.2	Dynamic Loading.....	3-17
3.9.3	Static Loading.....	3-17
3.10	Water Loads	3-17
	References	3-18
	Table 3-1 Component Loads	3-1
	Table 3-2 Component Loads for Preliminary Design Only	3-2
	Table 3-3 Material Classification.....	3-3
	Table 3-4 Equivalent Height of Soil for Calculating Live Load Surcharge	3-10
	Figure 3-1 Coulomb Theory	3-6

CHAPTER 3 - LOADS

Figure 3-2 Calculating β with Broken Backfill Surface.....3-7
Figure 3-3 Rankine Theory3-8
Figure 3-4 Seismic Performance Categories for Maine3-12

3 LOADS

3.1 General

The dead loads of the standard bridge components listed in Table 3-1 may be used for design computations or calculated separately at the option of the Structural Designer. The dead loads of the standard bridge components listed in Table 3-2 may be used for preliminary design only.

Table 3-1 Component Loads

Bridge Component	Design Load
Permanent Concrete Barrier Type IIIA	425 lb/ft
Permanent Concrete Barrier Type IIIB	600 lb/ft
2-Bar Steel Bridge Rail (without curb)	50 lb/ft
4-Bar Steel Bridge Rail - Traffic/Pedestrian (without sidewalk)	87 lb/ft
4-Bar Steel Bridge Rail - Traffic/Bicycle (without curb)	88 lb/ft
Texas Classic Bridge Rail - Traffic Rail (without curb)	300 lb/ft
Texas Classic Bridge Rail - Sidewalk Rail (without sidewalk)	371 lb/ft
Barrier Mounted Steel Bridge Railing - 1-Bar	9 lb/ft
Barrier Mounted Steel Bridge Railing - 2-Bar	19 lb/ft
3 inch bituminous wearing surface with membrane waterproofing	38 lb/ft ²
2 inch un-reinforced concrete wearing surface	25 lb/ft ²
Concrete Curb (20 inches wide with 3 inch bituminous wearing surface)	250 lb/ft
Concrete curb (20 inches wide with 2 inch concrete wearing surface)	220 lb/ft
Concrete curb with granite curb (24 inches wide with 3 inch bituminous wearing surface)	305 lb/ft
Concrete curb with granite curb (24 inches wide with 2 inch concrete wearing surface)	265 lb/ft

Bridge Component	Design Load
Concrete sidewalk 5' wide (includes concrete under bridge rail)	1110 lb/ft
Concrete sidewalk 6' wide (includes concrete under bridge rail)	1290 lb/ft
Diaphragms for rolled steel beam	15 lb/ft per beam
Diaphragms for welded steel plate girder	20 lb/ft per beam

3.2 MaineDOT Live Load Policy (New and Rehabilitation)

All new and replacement bridge-type structures should be designed by AASHTO LRFD. The live load used is the code-specified live load for all limit states except for Strength I. The Live Load used for the Strength I limit state is the Maine Modified Live Load which consists of the standard HL-93 Live Load with a 25% increase in the Design Truck. All buried structures should be designed by LFD with an HS25 truck in accordance with the AASHTO Standard Specifications.

The magnitude of the design live load to be used in rehabilitating existing structures should be determined in each individual case, taking into account the inherent strength of the existing structure and the cost involved in providing additional load carrying capacity. In general, such structures should be strengthened to at least the code specified HL-93 live load for all limit states. A design capacity less than HL-93 must be approved by the Engineer of Design.

The optional deflection criteria (AASHTO LRFD Section 2.5) should be checked by the Structural Designer using the standard HL-93 Live Load.

Load modifiers specified in AASHTO LRFD Section 1.3 relating to ductility and redundancy should generally be taken as 1.0. The use of non-ductile or non-redundant components is not allowed. The load modifier relating to operational importance should be taken as 1.0, unless otherwise indicated by the Engineer of Design.

3.3 Thermal Effects

The temperature range used to determine thermal forces and movements should be in conformance with the AASHTO LRFD "cold climate" temperature range.

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

3.6.2 Presence of Water

Retained earth should be drained and the development of hydrostatic water pressure eliminated by the use of a free-draining backfill such as crushed rock (less than 5 percent passing a No. 200 sieve), gravel drains, or other drainage systems. If retained earth is not allowed to drain, or if the groundwater levels differ on opposite sides of the wall, the effect of hydrostatic water pressure should be added to the earth pressure. Pore water pressures should be added to the effective horizontal stresses in determining total lateral earth pressure on the wall.

Walls along a stream or river should be designed for a minimum differential water pressure due to a 3 foot head of water in the backfill soil above the weepholes.

3.6.3 Earthquake

Where applicable, the effects of wall inertia and amplification of active earth pressure by earthquake should be considered. The Mononobe-Okabe method should be used to determine equivalent static pressures for seismic loads on walls and abutments as described in Section 3.7.3 Substructure. If the soils are saturated, liquefaction should be evaluated and addressed per Section 3.7.4.2 Liquefaction and Seismic Settlement.

3.6.4 Lateral Earth Pressure

The lateral earth pressure is linearly proportional to depth and is taken as:

$$\sigma = K \cdot \gamma_s \cdot z$$

where:

σ = lateral earth pressure at a given depth, z.

K = coefficient of lateral earth pressure, to be taken as:

K_a , *active*, for walls that move or deflect sufficiently to reach the active conditions (refer to Figure 3-1)

K_o , *at rest*, for walls that do not deflect or are restrained from movement

K_p , *passive*, for walls that deflect or move sufficiently to reach a passive condition, including integral abutments.

γ_s = soil unit weight (refer to Table 3-3)

z = depth

The resultant lateral earth load due to the weight of the backfill should be assumed to act at a height of H/3 above the base of the wall, where H is the total wall height, measured along a vertical plane extending from the ground surface above the back of the footing down to the bottom of the footing.

For walls with a total wall height, H, greater than or equal to 5 feet, the horizontal movement of the top of the wall due to structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

At-rest earth pressures are usually limited to bridge abutments to which superstructures are fixed prior to backfilling (e.g. rigid frame bridges) or to cantilever walls where the heel is restrained and the base/stem connection prevents rotation of the stem.

3.6.5 Active Earth Pressure Coefficient

3.6.5.1 Coulomb Theory

The Coulomb theory should be used for the design of the following yielding walls:

- Gravity shaped walls and abutments
- Semi gravity walls
- Prefabricated modular walls with steep back faces (20° or less measured from the vertical)
- Cantilever walls and abutments with short heels (refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of short heel)

In all of these cases, interface friction (δ) develops along the back face of the wall. For horizontal or sloped backfill surfaces, the value of the coefficient of active lateral earth pressure (Coulomb), K_a , may be taken as:

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \cdot \sin(\alpha - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)}} \right)^2}$$

where:

- α = angle (degrees) of backface of wall to the horizontal, as shown in Figure 3-1.
- ϕ = angle of internal soil friction (degrees), taken from Table 3-3.
- δ = friction angle (degrees) between fill and wall, taken from Table 3-3 for soil against concrete.
- β = angle of backfill to the horizontal (degrees), as shown in Figure 3-1.

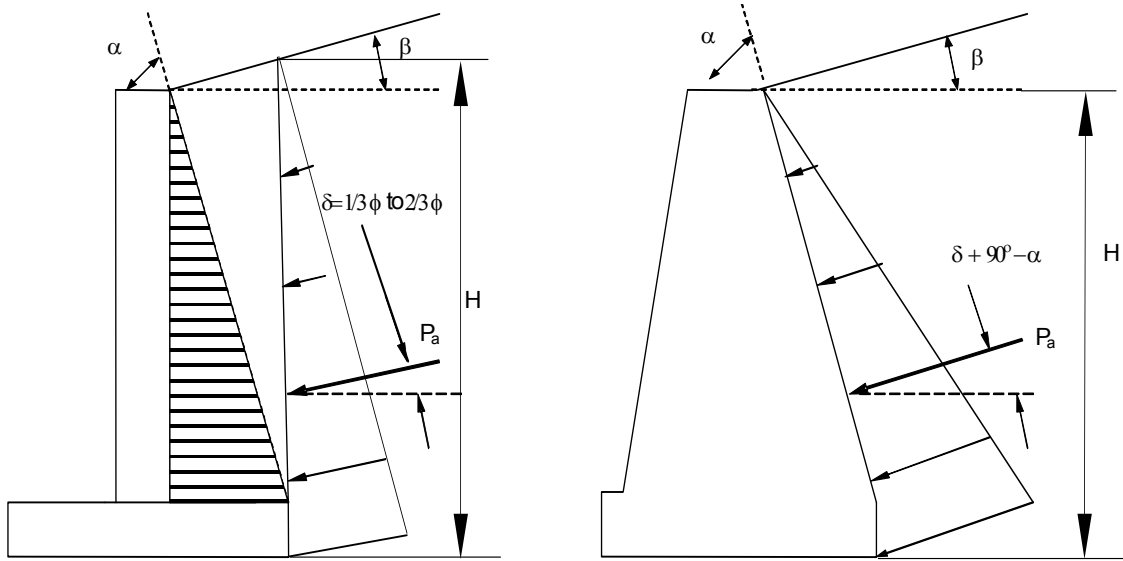


Figure 3-1 Coulomb Theory

The resultant earth pressure force, P_a , is oriented at an angle, either δ or $\delta + 90^\circ - \alpha$, as shown in Figure 3-1. The resultant acts at a distance, $H/3$, from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient, K_a , may be determined using a β value adjusted per AASHTO LRFD Figure 3.11.5.8.1-3 or substituted with β^* , as shown in Figure 3-2.

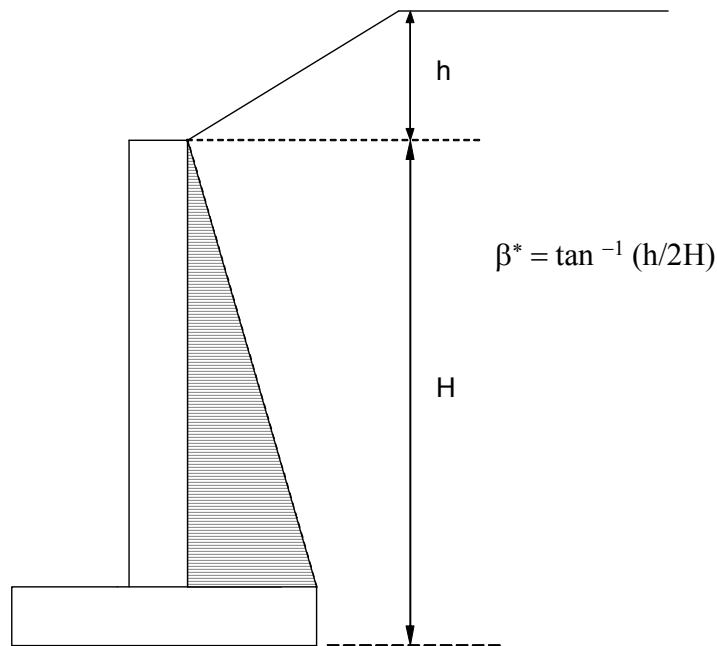


Figure 3-2 Calculating β with Broken Backfill Surface

Rankine theory, as described in Section 3.6.5.2, may also be used for the design of yielding walls, for a simplified analysis (at the Structural Designer's option). The use of Rankine theory will result in a slightly more conservative design.

3.6.5.2 Rankine Theory

Rankine theory should be used for long-heeled cantilever walls. Refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of a long heeled cantilever wall. For simplicity (at the Structural Designer's option), Rankine theory may also be used to compute lateral earth pressures on any yielding wall listed in 3.6.5.1 Coulomb Theory, although its use will result in a slightly more conservative design.

For these cases, interface friction between the wall backface and the backfill is not considered. Rankine earth pressure is applied to a plane extending vertically from the heel of the wall base, as shown in Figure 3-3.

For a horizontal backfill surface where $\beta = 0^\circ$, the value of the coefficient of active earth pressure (Rankine), K_a , may be taken as:

$$K_a = \tan^2 \left(45^\circ - \frac{\phi}{2} \right)$$

where:

ϕ = angle of internal soil friction (degrees), taken from Table 3-3.

β = angle of backfill to the horizontal (degrees), as shown in Figure 3-3.

For a sloped backfill surface where $\beta > 0^\circ$, the coefficient of active earth pressure (Rankine), K_a , may be taken as:

$$K_a = \cos \beta \cdot \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

- δ = friction angle between fill and wall (degrees), taken from Table 3-3 for soil against concrete.
- β = angle of backfill to the horizontal (degrees), as shown in Figure 3-1.

The resultant passive earth pressure force, P_p , is oriented at an angle, δ , to the normal drawn to the back face of the wall. The resultant passive earth load should be assumed to act at a distance of $H/3$ measured from the bottom of the footing.

3.6.7 Lateral Earth Pressures for Unconventional Retaining Walls

3.6.7.1 Mechanically Stabilized Earth Walls

For mechanically stabilized earth (MSE) walls, the resultant earth pressure, P_a , should be calculated using the active earth pressure coefficient, K_a , as described in Section 3.6.5.1 Coulomb Theory. For sloping and broken backfill surfaces, earth pressures should be calculated per AASHTO LRFD Figures 3.11.5.8 - 1 thru 3.

3.6.7.2 Prefabricated Modular Walls

This category includes prefabricated concrete modular gravity (PCMG) walls, metal bin walls, and gabion walls. Where the back of the prefabricated modules form an irregular stepped surface, the earth pressure should 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 Rankine earth pressure theory. The magnitude and location of the resultant earth loads may be determined using the earth pressure distributions shown in AASHTO LRFD Figures 3.11.5.9 -1 and 2.

3.6.8 Surcharge Loads – Live Load Surcharge

A live load surcharge should be applied when traffic loads are located within a horizontal distance equal to one-half of the wall height, H , behind the back of the wall. H is defined as the total wall height, measured along a vertical plane extending from the bottom of the footing up to the ground surface at the back of the wall. The additional lateral earth pressure due to live load should be modeled by a surcharge load equal to that applied by a height of soil, H_{eq} , defined in Table 3-3. The surcharge will result in the application of an additional uniform, constant horizontal pressure on the back of the wall having a magnitude P_s , taken as:

$$P_s = H_{eq} \cdot \gamma_s \cdot K$$

where:

- P_s = constant horizontal pressure due to live load surcharge
- γ_s = soil unit weight of soil, taken as 125 lb/ft³
- K = coefficient of lateral earth pressure, K, as defined in Section 3.6.4 Lateral Earth Pressure
- H_{eq} = equivalent height of soil for live load surcharge, determined from Table 3-4

The resultant horizontal earth pressure due to live load surcharge acts at the mid-height of the wall. The wall height is taken as the distance between the surface of the backfill and the bottom of the footing.

Table 3-4 Equivalent Height of Soil for Calculating Live Load Surcharge

Abutment or Wall Height (ft)	H_{eq} (ft), Edge of Traffic is Normal to Wall or Abutment	H_{eq} (ft), Edge of Traffic is Parallel to Wall and Located at Back of Wall	H_{eq} (ft), Edge of Traffic is Parallel to Wall and Located 1 ft or More from Back of Wall
3	4	5	2
10	3	3.5	2
≥ 20	2	2	2

Note: Linear interpolation should be used for intermediate wall heights.

3.6.9 Passive Earth Pressure Loads

The resistance due to passive earth pressure in front of walls should be neglected unless the wall extends well below the depth of frost penetration, scour, or other types of potential disturbance, such as utility trench excavation in front of the wall. Neglecting this passive earth pressure is due to the consideration that the soil may be removed during future construction, which will eliminate its contribution to wall stability.

3.7 Seismic

3.7.1 General

The following criteria will be used to determine the scope of seismic analysis required.

3.7.1.1 Seismicity of Site

According to AASHTO Standard Specifications Division I-A, Maine has a relatively low seismic risk. From Figure 3-4, it is noted that a portion of

northern Maine is bounded by an isoseismal of $A = 0.10g$. Bridges located in areas where the horizontal acceleration coefficient is less than or equal to 0.09 will be assigned to Seismic Performance Category (SPC) A. Bridges located in areas where $0.09 < A < 0.19$ will be assigned to SPC B.

AASHTO Standard Specifications Division I-A has not clearly defined the location of the 0.09 isoseismal for Maine, but Figure 3-4 provides this information. In this figure, an interpretation of the location of the 0.09 isoseismal was made through information provided by the Maine Geologic Survey. In general, SPC B will require a higher level of seismic performance analysis than SPC A.

3.7.1.2 Geotechnical Characteristics of the Site

Soil conditions must be known to determine the seismic site coefficient for the bridge. In the AASHTO Standard Specifications Division I-A there are four soil profiles defined and a site coefficient is assigned to each profile. Additionally, potential hazards and seismic design requirements related to slope stability, liquefaction, fill settlement, and any increase in lateral earth pressures as a result of earthquake motion need to be identified. If required, the Geotechnical Designer will provide recommendations for site stabilization and design earth pressures.

3.7.1.3 Functional Importance

Bridges located on the NHS should be recognized as essential. Refer to AASHTO Standard Specifications Division I-A Section 3.3.

3.7.1.4 Major or Minor Structures

Bridges are divided into two groups based on economics. Major bridges will be defined as those with bridge construction costs in excess of \$10 million. All other bridges will be considered minor bridges.

3.7.1.5 Structure Type and Detail

Certain bridge types (e.g. multiple simple spans), or details (e.g. high rocker bearings) that are more vulnerable to earthquake damage should be avoided based on the probable severity of damage and the impact on the serviceability of the structure.

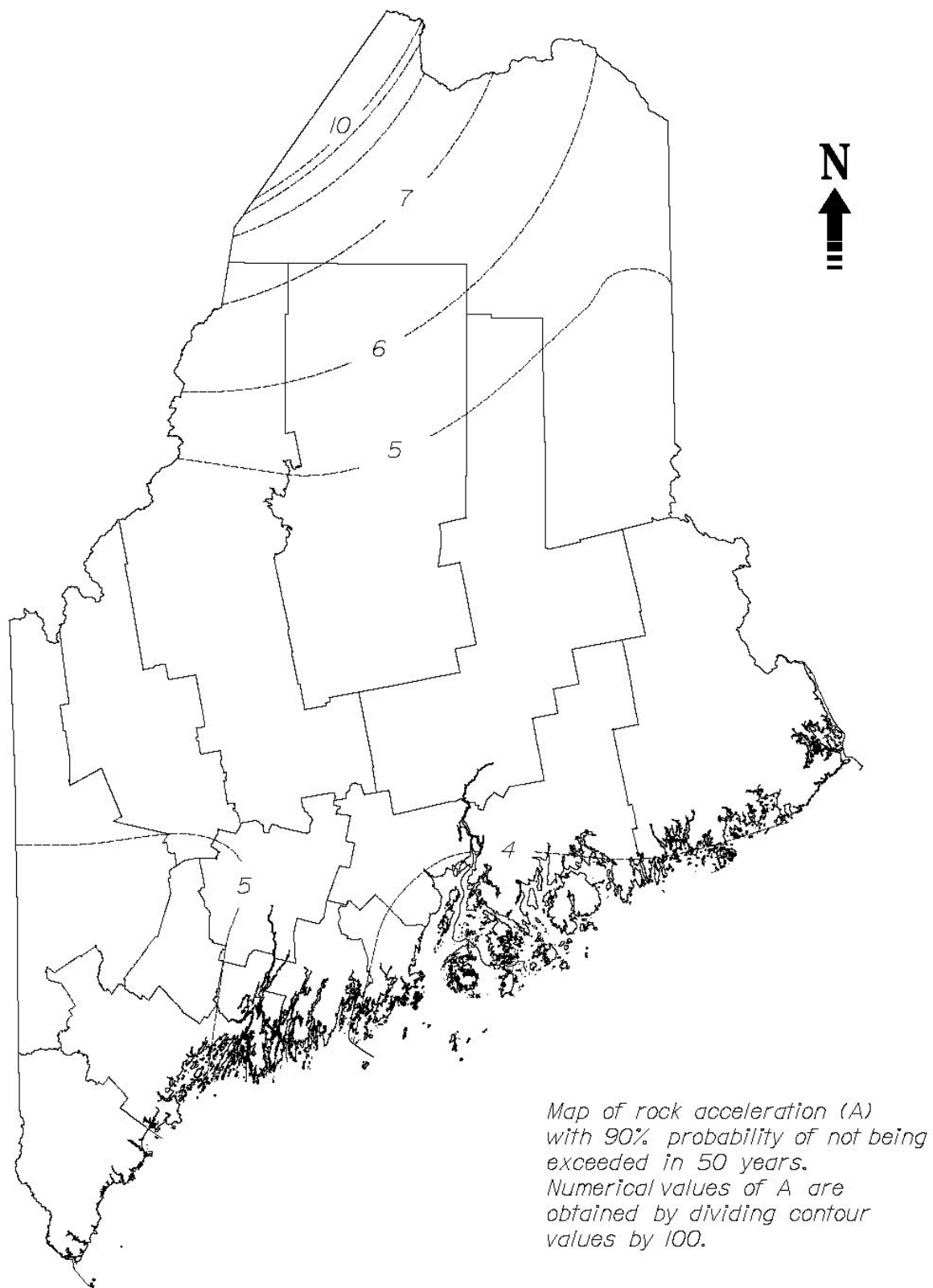


Figure 3-4 Seismic Performance Categories for Maine

Some other special conditions that are particularly sensitive to seismic forces are as follows:

- Single or individual column pier supports
- High, slender pier columns (where the slenderness ratio exceeds 60)
- Large skews (generally in excess of 45 degrees) with substandard support lengths
- Severe curvature where the subtended arc exceeds 75 degrees
- Unusual geometry causing portions of the structure to be significantly different in stiffness or that results in unusual support or framing details
- Hinges or seated connections in suspended superstructures
- Non load-path redundant superstructures

3.7.2 Seismic Analysis

Analysis is done based on two categories:

- o *SPC A Bridges:* AASHTO Standard Specifications Division I-A Section 4.5 indicates that for SPC A, no detailed analysis is required other than connection design and bearing seat length. For the MaineDOT Bridge Program, this will be amended such that all major and functionally important bridges (with two or more spans) in SPC A will be designed according to the requirements for SPC B with an acceleration coefficient of $A = 0.09$.
- o *SPC B Bridges:* For SPC B bridges with two or more spans, a detailed seismic analysis is required. AASHTO Standard Specifications Division I-A Section 4.2 indicates that a single mode spectral analysis is adequate for both "regular" and "irregular" bridges. A "regular" bridge is defined as one having no abrupt or unusual changes in mass, stiffness, or geometry along its length, and no large differences in these quantities (>25%) between adjacent supports. "Irregular" bridges are ones that do not satisfy the definition of "regular" bridges.

Structural Designers should make every attempt to avoid designing "irregular" bridges or bridges with special conditions and should adopt good structural form where possible. The basics of good structural form are:

- o *Simplicity* - it is best to ensure that the transfer of loads is by the shortest and simplest route possible.
- o *Symmetry* - seismic loads are inertial loads and act through the center of mass of each component while the resultant of the resisting force acts through the center of stiffness. In order for a bridge deck not to rotate, the eccentricity between the applied force and the resisting force should be zero. Symmetry requires that the various sources of lateral stiffness in a bridge (i.e., the piers and the abutments) be symmetrically located about the center of mass.
- o *Integrity* - This means that the various components of a bridge must remain connected together during an earthquake. Careful detailing is important. Generous girder seating lengths, conservative bearing details, confining steel in plastic zones, generous rebar anchorage lengths, shear keys, and other restraining devices are all examples of measures that will ensure a structure's integrity for seismic loads.

"Seismic Design and Retrofit Manual for Highway Bridges" (FHWA 1987) located in the Bridge Design Library, has several examples of acceptable structural form (refer to Chapter 5 Substructures). Structural Designers should refer to this and use it as a guide to design. Where it is impossible to avoid a structure that is "irregular" and located in SPC B, this manual recommends that a multi-modal method of analysis be done. This is because regular bridges are assumed to respond to earthquake loads in a single or fundamental mode of deformation. This is a reasonable assumption for regular, uniform structures, but may be in gross error for more complex structures. Irregular bridges can vibrate in other mode shapes besides the fundamental mode shape and still satisfy equilibrium. Irregular or unusual bridges are also likely to have higher modes, which will need to be considered.

The AASHTO Standard Specifications Division I-A provides guidelines on how to perform a single mode analysis. This method can be done manually using hand procedures or by computer methods. Usually the latter is preferred for all but the simplest bridges. General purpose space frame programs are capable of doing a single mode analysis through the use of the uniform load method.

3.7.3 Substructure

The recommended method of analysis of substructure units for seismic loads is described in Article 7.4.3 of AASHTO Standard Specifications Division I-A and the Specification Seismic Design Commentary. Additional guidance is provided in "Design Examples 1 through 7" (FHWA1997).

The recommended procedures include applying the Mononobe-Okabe Method of analysis for lateral earth overpressure, and accounting for the seismic

inertia forces of both the substructure self weight and the soil resting on the substructure footings. The earthquake overpressure force is equal to the total active earth pressure force (including seismic) as calculated by AASHTO Standard Specifications Division 1-A Equation C6-3, less the active (static) earth pressure. The earthquake overpressure force includes only the additional seismic pressure that occurs during an earthquake. The centroid of this additional force is assumed to act at a distance of 0.6H above the top of the footing.

3.7.4 Embankments & Embankments Supporting Substructure Units

3.7.4.1 Seismic Slope Stability

Seismic stability of slopes and embankments is evaluated using the Unified Methodology for Seismic Stability and Deformation Analysis. Refer to Chapter 7 of "Geotechnical Earthquake Engineering for Highways" (FHWA, 1997).

The Unified Methodology combines two accepted methods for seismic stability: the seismic coefficient-Factor of Safety (FOS) approach and the permanent seismic deformation approach. First, a seismic coefficient FOS analysis is performed. Then, if the seismic coefficient FOS analysis results in a factor of safety less than 1.0, a permanent seismic deformation analysis is performed.

A variety of computer programs can be used to perform both of these pseudo-static limit equilibrium analyses: PCSTABL4, PCSTABLE5, XSTABLE, and SLOPEW. Seismic loads depend on the bedrock acceleration at the site, and a seismic coefficient. Consult "Geotechnical Earthquake Engineering for Highways" (FHWA, 1997) for guidance on selection of a seismic coefficient. The Site Coefficient Factors (SCF) in the AASHTO Standard Specifications are for the structural and geotechnical analysis of walls and bridge foundations and are not applicable to slope stability and liquefaction analyses.

3.7.4.2 Liquefaction and Seismic Settlement

Liquefaction potential should be assessed employing the Simplified Procedure, originally developed by Seed and Idriss (1982) and progressively refined and summarized (FHWA, May 1997). For soil units in which the factor of safety against initial liquefaction is unsatisfactory, a liquefaction impact analysis must demonstrate that the site will still perform adequately if liquefaction occurs. Potential impacts of liquefaction include bearing capacity failure, loss of lateral support for piles, lateral squeezing, post-liquefaction-induced settlement, and downdrag. Liquefaction-induced

settlement of unsaturated sands must also be addressed as part of the post-liquefaction assessment (Tokimatsu and Seed, 1987).

If the seismic impact analyses yield unacceptable deformations, consideration may be given to performing a more sophisticated liquefaction potential assessment and to evaluation of liquefaction potential mitigation measures.

3.8 Non-Vehicular Bridges

The design of prefabricated non-vehicular bridges should be in general accordance with the AASHTO "Guide Specification for Design of Pedestrian Bridges." Pedestrian bridges that are not prefabricated, long spans, or non-typical should be designed according to AASHTO LRFD Specifications. The optional deflection criteria and the use of load modifiers should be in accordance with Section 3.2 MaineDOT Live Load Policy (New and Rehabilitation).

The design live and dead loads of the bridge should be determined by considering several issues. For live loads, consider the width of the bridge, vertical clearance, use by emergency and maintenance vehicles, and use by snow grooming equipment. Dead loads should consider the type of rail, the use of a rub rail, security fencing, lighting, and any utilities (present or future). For further guidance on the applicability of dead and live loads, refer to Section 1.6 Non-Vehicular Bridges.

In general, a 10 foot wide non-vehicular bridge should be designed for the appropriate pedestrian load and an H5 (10,000 pound vehicle with 2,000 pound front axle and 8,000 pound rear axle) vehicular live load. The Structural Designer should be aware that some snowmobile grooming equipment can weigh close to 15,000 pounds with a distributed dead load of up to 400 pounds per square foot.

3.9 Ice Loads

3.9.1 General

The following criteria are to be used when designing for ice loads. Static loading should be used when it is anticipated that ice may occur between two substructure units while having open water in an adjacent span. Static ice loads should be applied separately and not combined with dynamic ice loads. It is not necessary to design for uplift or ice jams except in very special circumstances.

3.9.2 Dynamic Loading

The north/south zone boundary passes through Rangeley, Guilford, Medway, and Houlton.

- o *Design Pressure:* 200 psi on pier nose @ Q1.1
100 psi on pier nose @ Q50
- o *Coefficients:* Apply nose inclination, pier width, and ice thickness factors given in AASHTO LRFD
- o *Ice Thickness:* 2 feet in northern zone
1'-6" in southern zone

Add 6 inches when ice conditions are known to be severe. Rivers known to have severe ice conditions are the St. John, Allagash, Aroostook, Penobscot, Kennebec, and Androscoggin Rivers
- o *Transverse Force:* 30 percent of longitudinal force
- o *Point of Application:* Q50 or Q1.1 elevation

3.9.3 Static Loading

- o *Design Load:* 5 k/ft on pier side
- o *Point of Application:* Q1.1 elevation

3.10 Water Loads

Static water pressure should be determined in accordance with AASHTO LRFD Section 3.7.1. Consideration should be given to the following design water levels for all limits states:

- o *Design flood event* – Q50
- o *Normal high water* – Q1.1

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Chapter 4

SUPERSTRUCTURES



Sagadahoc Bridge, Bath-Woolwich



Kittery Point Bridge, Kittery

4	SUPERSTRUCTURES	4-1
4.1	Bridge Widths	4-1
4.1.1	General.....	4-1
4.1.2	Urban Bridge Widths	4-1
4.1.3	Rural Bridge Width Standards Tables	4-1
4.1.3.1	New or Reconstructed Bridges.....	4-2
4.1.3.2	Bridges to Remain in Place	4-6
4.2	Lighting.....	4-7
4.3	Curbs and Sidewalks.....	4-7
4.4	Bridge Rail	4-8
4.4.1	Definitions.....	4-8
4.4.2	General.....	4-9
4.4.2.1	TL-2 Rails	4-10
4.4.2.2	TL-4 Rails	4-10
4.4.2.3	TL-5 Rails	4-11
4.4.3	Bridge Rail Selection	4-11
4.4.4	Bicycle Railing	4-17
4.4.5	Reduced Standard Bridge Rail	4-18
4.4.6	Aesthetics.....	4-18
4.4.7	Transitions.....	4-18
4.5	Security Fences.....	4-19
4.6	Wearing Surfaces	4-20
4.6.1	General.....	4-20
4.6.2	Descriptions.....	4-20
4.6.2.1	Bituminous Wearing Surface with Membrane	4-20
4.6.2.2	Unreinforced Structural Concrete Wearing Surface	4-20
4.6.2.3	Integral Concrete Wearing Surface	4-20
4.7	Membranes.....	4-21
4.8	Deck Joints and Expansion Devices.....	4-21
4.8.1	General.....	4-21
4.8.2	Preformed Elastomeric Joint Seals.....	4-22
4.8.3	Gland Seals.....	4-24
4.8.4	Finger Joints.....	4-24
4.8.5	Modular Joints	4-25
4.8.6	Design/Contract Drawings Presentation.....	4-28
4.9	Drains	4-28
4.9.1	Design	4-28
4.9.2	Use of Standard Details.....	4-29
4.9.3	Bridge Drain Spacing Tables.....	4-29
4.10	Utilities	4-52
4.11	Bearings	4-53
4.11.1	General.....	4-53
4.11.2	Elastomeric Pads.....	4-54
4.11.3	Steel-Reinforced Elastomeric Bearings	4-54
4.11.3.1	Design Rotation.....	4-54
4.11.3.2	Design Movement	4-55

CHAPTER 4 - SUPERSTRUCTURE

4.11.3.3	Masonry Plates.....	4-55
4.11.3.4	Sole Plate Details.....	4-55
4.11.3.5	Bearing to Girder Connection.....	4-55
4.11.3.6	Lateral and Uplift Restraint.....	4-56
4.11.3.7	Anchor Rods.....	4-56
4.11.3.8	Elastomeric Bearings with Sliding Surfaces.....	4-56
4.11.3.9	Marking.....	4-57
4.11.4	High Load Multi-Rotational Bearings.....	4-57
4.11.4.1	Design Recommendations.....	4-57
4.11.4.2	Uplift Forces.....	4-57
4.11.4.3	Design Rotation.....	4-58
4.11.4.4	Design Movement.....	4-58
4.11.4.5	Sole Plate.....	4-58
4.11.4.6	Future Maintenance.....	4-58
4.11.4.7	Masonry Plate and Anchor Rods.....	4-58
4.11.4.8	Marking.....	4-58
4.11.4.9	Pot Bearings and Disk-type Bearings.....	4-59
4.11.5	Steel Bearings.....	4-59
	References.....	4-60
	Table 4-1 Bridge Roadway Width Standards – Rural NHS.....	4-2
	Table 4-2 Bridge Roadway Width Standards – Rural Non-NHS.....	4-3
	Table 4-3 Bridge Roadway Width Standards – Special Purpose Roads.....	4-5
	Table 4-4 Bridges to Remain in Place – Rural Non-NHS.....	4-6
	Table 4-5 Bridge Rail Test Equivalencies.....	4-9
	Table 4-6 Bridge Rail Performance Level Selection.....	4-16
	Table 4-7 Elastomeric Joint Seal Movement Ratings.....	4-23
	Table 4-8 Bridge Drain Type A.....	4-31
	Table 4-9 Bridge Drain Type A.....	4-32
	Table 4-10 Bridge Drain Type A.....	4-33
	Table 4-11 Bridge Drain Type A.....	4-34
	Table 4-12 Bridge Drain Type A.....	4-35
	Table 4-13 Bridge Drain Type A.....	4-36
	Table 4-14 Bridge Drain Type A.....	4-37
	Table 4-15 Bridge Drain Type B.....	4-38
	Table 4-16 Bridge Drain Type B.....	4-39
	Table 4-17 Bridge Drain Type B.....	4-40
	Table 4-18 Bridge Drain Type B.....	4-41
	Table 4-19 Bridge Drain Type B.....	4-42
	Table 4-20 Bridge Drain Type B.....	4-43
	Table 4-21 Bridge Drain Type B.....	4-44
	Table 4-22 Bridge Drain Type C.....	4-45
	Table 4-23 Bridge Drain Type C.....	4-46
	Table 4-24 Bridge Drain Type C.....	4-47
	Table 4-25 Bridge Drain Type C.....	4-48
	Table 4-26 Bridge Drain Type C.....	4-49

CHAPTER 4 - SUPERSTRUCTURE

Table 4-27 Bridge Drain Type C	4-50
Table 4-28 Bridge Drain Type C	4-51
Figure 4-1 TL-2 Timber Rail Systems	4-12
Figure 4-2 K_g and K_c	4-14
Figure 4-3 K_s	4-15
Figure 4-4 Curtain System	4-26
Figure 4-5 Trough System	4-27
Procedure 4-1 Rail Design	4-13
Example 4-1 Compression Seal Design	4-24

4 SUPERSTRUCTURES

4.1 Bridge Widths

4.1.1 General

Section 4.1 is a guide to determine the appropriate width of a bridge. For geometric design criteria for the width of the approaches to the bridge, refer to Section 2.8.1 Roadway Widths. For pedestrian and bicycle considerations, refer to Section 4.3 Curbs and Sidewalks and Section 4.4.4 Bicycle Railing.

As discussed in Section 2.8.1 Roadway Widths, all roads are classified according to function and further divided into two categories, urban and rural. For rural roads, bridge widths are given in Table 4-1 through Table 4-4. Bridges to Remain in Place are defined as those in which the substructure will remain in place without widening. Special Purpose Roads are a special type of local road that is defined further in Section 4.1.3.1C.

4.1.2 Urban Bridge Widths

For urban streets, the clear width for new bridges should be the same as the width of approaches. For bridges greater than 200 feet long on streets that have shoulders wider than 4 feet, the shoulders may be reduced to 4 feet on each side.

4.1.3 Rural Bridge Width Standards Tables

Table 4-1 through Table 4-4 are used as starting points for selecting bridge widths. Route continuity should be considered when determining widths for each project. Established or proposed corridor widths should be obtained from the Bureau of Planning. For bridges on local roads, proactive communication with municipal officials and/or planning boards will identify established or reasonable proposed corridor widths.

In addition to AADT and corridor width, other factors should be investigated before a final bridge width is determined. They include, but are not limited to, geometry, accident history, right-of-way and environmental impacts, archeological and historic concerns, local impacts, bicycle and pedestrian use, and cost.

Bridge widths are curb-to-curb or rail-to-rail, whichever is less. Exceptions to the bridge width standards must be obtained from the Engineer of Design. Exceptions to the bridge width standards on NHS roadways must also be obtained from FHWA.

4.1.3.1 New or Reconstructed Bridges

A. NHS

**Table 4-1 Bridge Roadway Width Standards – Rural NHS
New or Reconstructed Bridges**

Design Traffic AADT	Design Speed (mph)	Traveled Way (ft)	Bridge Width ¹ (ft)
< 400	40-55	22 ²	30
< 400	60-75	24 ³	32
400-1500	40-55	22 ²	34
400-1500	60-75	24 ³	36
1500-2000	40-45	22 ²	34
1500-2000	50-75	24 ³	36
> 2000	40-75	24 ³	40

1. Bridges greater than 200 feet long may have a reduced bridge width equal to the traveled way plus 4 foot shoulders on each side.
2. The traveled way pavement thickness should be paved full depth for a full 24 foot width.
3. Traveled way widths of 22 feet may be used if alignment and safety records are deemed satisfactory, and the existing corridor has a 22 foot traveled way width. Bridge widths should be reduced accordingly.

B. Non-NHS

**Table 4-2 Bridge Roadway Width Standards – Rural Non-NHS
New or Reconstructed Bridges**

Local Roads and Minor Collectors ¹

Design Traffic AADT	Design Speed (mph)	Traveled Way (ft)	Bridge Width (ft)
< 1000	40 ³	22 ^{3,4}	28 ⁴
1000-4000	40 ³	22 ^{5,6}	30 ^{4,6,9}
> 4000	Refer to Major Collectors		

Major Collectors ¹

Design Traffic AADT	Design Speed (mph)	Traveled Way (ft)	Bridge Width (ft)
< 1000	45	22 ⁵	28
1000-4000	45	22 ⁶	30 ^{6,9}
4000-6000	45	22	34
6000-8000	45	24 ⁷	36 ⁷
> 8000	55	24 ⁸	40 ⁸

Minor Arterials ²

Design Traffic AADT	Design Speed (mph)	Traveled Way (ft)	Bridge Width (ft)
< 1000	45	22 ⁵	28
1000-4000	45	22 ⁶	30 ^{6,9}
4000-6000	45	22	34
6000-8000	55	24 ⁷	36 ⁷
> 8000	55	24	40

1. Bridges located on local roads and all collectors greater than 100 feet long may have a reduced bridge width equal to the traveled way plus 4 foot shoulders on each side.
2. Bridges located on minor arterials greater than 200 feet long may have a reduced bridge width equal to the traveled way plus 4 foot shoulders on each side.
3. The Designer should scrutinize the design speeds for bridges on local roads on each project for the best fit in the local area.
4. In order to minimize impacts and costs and stay within the footprint of the existing highway, bridges on local roads that

have low speeds, good geometric characteristics, and low CRF may be considered for a decreased bridge width. When AADT < 250, 22 foot bridge widths may be considered. When AADT is between 250 and 750, 24 foot bridge widths may be considered. When AADT < 2000, 26 foot bridge widths may be considered.

5. When AADT is less than 1000, the traveled way width may be reduced to 20 feet, with bridge widths remaining at 28 feet. |
6. When AADT is 1000-4000, the traveled way width may be reduced to 20 feet, with bridge widths reduced to 28 feet.
7. When AADT is 6000-8000, the traveled way width may be reduced to 22 feet, with bridge widths reduced to 34 feet.
8. When AADT is greater than 8000, the traveled way width may be reduced to 22 feet, with bridge widths reduced to 36 feet. |
9. When the bridge rail and approach guard rail lengths are continuous for greater than 1000 feet on each side, and AADT is between 2000-4000, consideration should be given to widening the rail to rail width to 32 feet to minimize conflict for snowplowing operations.

C. Special Purpose Roads

Special purpose roads are generally lightly traveled and of low speed, deserving special design consideration. They include recreational roads, resource development roads, and local service roads. Resource development roads include mining and logging roads, and the criteria for recreational roads should be followed where applicable. Local service roads serve isolated areas that have little or no potential for further development. Most of these roads will dead-end at the service to the last parcel of land. Traffic is generally less than 100 AADT and is of a repeat type. The criteria for recreational roads should be followed where applicable.

**Table 4-3 Bridge Roadway Width Standards – Special Purpose Roads
New or Reconstructed Bridges**

Type	Traveled Way ¹ (ft)	Bridge Width (ft)
Primary Access Road (2-lane) ²	22-24	26-32
Circulation Road (2-lane) ³	20-22	24-30
Area Road (2-lane) ⁴	18-20	18-24
Area Road (1-lane) ^{4,5}	12	12-14

1. Widening on the inside of sharp curves should be provided. This additional width should be equal to 400 divided by the curve radius in feet.
2. Primary access roads are roads that allow through movement into and between access areas.
3. Circulation roads are roads that allow movement between activity sites within an access area.
4. Area roads are roads that allow direct access to individual activity areas such as campgrounds, park areas, boat launching ramps, picnic groves, scenic sites, and historic sites.
5. Area roads with AADT<100 may be designed as a two-way, single lane roadway. Roadway widths greater than 14 feet should not be used because drivers will tend to use the facility as a two-lane road.

4.1.3.2 Bridges to Remain in Place

A. Arterials (NHS and Non-NHS)

For an existing bridge to remain in place on an arterial, it should have adequate structural strength and a bridge width equal to the approach traveled way width plus 2 feet on each side. Bridges on arterials should be considered for ultimate widening or replacement if they do not provide at least HL-93 loadings. As an interim measure, narrow bridges should be considered for special narrow bridge treatments such as signing and pavement marking.

B. Collectors and Local

**Table 4-4 Bridges to Remain in Place – Rural Non-NHS
Minimum Bridge Widths**

Design Traffic AADT	Local^{1,2,3} (ft)	Collector³ (ft)
< 250	20	22
250-1500	22	22
1500-2000	24	24
> 2000	28	28

1. On local roads with few trucks, widths may be reduced by 2 feet. In no case shall the minimum clear width be less than the approach traveled way width.
2. Bridges on local roads with AADT<50 may be considered for one lane. The preferred width is equal to the traveled way width plus 1 foot shoulders on each side, with a minimum bridge width of 14 feet and maximum bridge width of 18 feet.
3. The values do not apply to bridges greater than 100 feet in total length. These bridges should be analyzed individually, taking into consideration the clear width provided, traffic volume, remaining life of the structure, pedestrian volume, snow storage, design speed, accident record, and other pertinent factors.

4.2 Lighting

MaineDOT, via the project team, will recommend to a municipality that a bridge should be lighted when it would be in the best interests of the public. The policy for the overhead lighting of bridges is as follows:

- o *Controlled Access Highways.* Bridges will be lighted when they are part of an interchange, where continuous lighting exists, and/or a need for lighting is established during preliminary design.
- o *Non-Compact Areas.* A bridge in a non-compact area will only be considered for lighting when requested by the municipality and a need for lighting is established during preliminary design.
- o *Compact Areas.* A bridge in a compact area should be lighted when at least one of the following is true:
 1. Lighting is requested by the municipality.
 2. Both approaches are lighted.
 3. There is significant pedestrian movement.
 4. Other safety issues are identified.

The cost of the installation of the light standards, foundations, and conduits are borne by MaineDOT on all bridges. The installation and maintenance of the wiring and the luminaries, as well as the cost of electricity, are the responsibility of the municipality, except on controlled access highways where MaineDOT is responsible.

MaineDOT's Traffic Engineering will determine the need and design for lighting under grade separation structures. The location of fixtures and level of illumination should be coordinated with Traffic Engineering, in accordance with "An Informational Guide for Roadway Lighting" (1984).

The Bridge Program is responsible for executing this lighting policy. Lighting needs should be documented in the PDR. Municipalities should be contacted as appropriate, and the City/State Agreement should appropriately reflect the arrangement.

4.3 Curbs and Sidewalks

For standard steel bridge railing, bridge curb reveal is generally 9 inches. Refer to Standard Detail 502 (03) for further guidance. On relatively short urban bridges, a reduced curb reveal can be considered to match approach curbs. Traffic railings should be flush with the face of curb except when calling for

granite bridge curbing, where the face of curb will project 5 inches in front of the face of rail.

The need for sidewalks should be considered on a project-by-project basis. Sidewalks should be included on a bridge when there are sidewalks on the approaches, or when it is determined that a sidewalk is warranted. A sidewalk should be included on either one or both sides of a bridge located in or adjacent to village areas or located near pedestrian generators such as neighborhoods, schools, businesses, and commercial development areas. The MaineDOT Bicycle and Pedestrian Coordinator can help determine whether a sidewalk is warranted, based on the criteria for Evaluating Existing or Potential Pedestrian Demand found in the MaineDOT Municipal/Local Cost Sharing Policy.

When MaineDOT determines that a sidewalk does not meet Category 1 (fully funded) of the Municipal/Local Cost Sharing Policy, a municipality may request that a sidewalk be provided. In this case, the municipality will be required to pay either 50% or the full cost of providing the sidewalk, in accordance with the criteria outlined in the policy.

Sidewalks with projected minimal pedestrian traffic should be 5 feet clear to the face of rail. Sidewalks with projected significant pedestrian traffic should be 6 feet clear to the face of rail. Sidewalk widths for very high pedestrian traffic should be determined on a project-by-project basis. Traffic railings or barriers separating vehicular traffic from pedestrian traffic should be considered only for exceptional cases. Sidewalks with no separation between pedestrian and vehicular traffic will require a combination pedestrian/traffic rail.

Wide sidewalks may hinder bridge inspection activities which use the under bridge crane. Bridge Maintenance should be consulted before proposing a sidewalk width greater than 6 feet.

Granite bridge curbing may be used only where granite curbing is called for on both approaches. In all other cases, curbs and sidewalks should be entirely concrete with a 1 inch batter of the face of the curb.

Concrete for curbs and sidewalks is Class LP.

4.4 Bridge Rail

4.4.1 Definitions

The following definitions are used when selecting a rail system.

- Adjusted ADT: ADT_{cy} adjusted for site condition criteria
- ADT_{cy}: average daily traffic for construction year

CHAPTER 4 - SUPERSTRUCTURES

- K_c : adjustment factor for horizontal curvature of alignment (refer to Figure 4-2)
- K_g : adjustment factor for grade (refer to Figure 4-2)
- K_s : adjustment factor for deck height and under structure conditions (refer to Figure 4-3)

- o *TL-1*: a test level 1 bridge rail generally acceptable for very low volume, low speed streets.
- o *TL-2*: a test level 2 bridge rail generally acceptable for most local and collector roads with favorable site conditions.
- o *TL-3*: a test level 3 bridge rail generally acceptable for a wide range of high speed arterial highways with very low mixtures of heavy vehicles and with favorable site conditions.
- o *TL-4*: a test level 4 bridge rail generally acceptable for the majority of applications on high-speed highways with a mixture of trucks and heavy vehicles.
- o *TL-5a*: a test level 5a bridge rail generally acceptable for the same applications as the *TL-4*, but when site conditions justify a higher level of rail resistance.
- o *TL-5 & TL-6*: test level 5 & 6 bridge rails generally acceptable on high speed, high volume highways with a higher ratio of heavy vehicles and unfavorable site conditions.

4.4.2 General

AASHTO LRFD Section 13 states that new bridge railings and the attachment to the deck overhang must satisfy crash-testing requirements to demonstrate compliance with structural and geometric requirements of a specified railing test level. These test levels are TL-1 through TL-6. The previous railing performance categories recognized by AASHTO were the PL-1 to PL-3 levels. Most bridges in Maine will require a TL-4 rail or less. Table 4-5 gives the accepted equivalency of the previously used crash-test parameters:

Table 4-5 Bridge Rail Test Equivalencies

Bridge Railing Testing Criteria	Accepted Equivalencies						
AASHTO LRFD	TL-1	TL-2	TL-3	TL-4	TL-5a	TL-5	TL-6
AASHTO Guide Specification		PL-1		PL-2		PL-3	
NCHRP Report 350	TL-1	TL-2	TL-3	TL-4		TL-5	TL-6
NCHRP Report 230		MSL-1 MSL-2*		MSL-3			

* MSL-2 is close to a TL-3 but adequate TL-3 performance cannot be assured without a pickup truck test.

To determine the TL level rail required, refer to Section 4.4.3 Bridge Rail Selection. To determine if a pedestrian or bicycle rail is required, refer to Sections 4.3 Curbs and Sidewalks and 4.4.4 Bicycle Railing.

The choice of which rail to use will generally be made from the following list, although other crash-tested rails are available through the FHWA website <http://safety.fhwa.dot.gov/fourthlevel/hardware/bridgerailings.htm>. Use of a rail from another state requires approval from the Engineer of Design, since additional design and detailing time will be necessary.

Commentary: In the late 1970's and early 1980's crash tests were conducted on several commonly used railings that had been designed according to the static load standards of the day. These tests resulted in the failure of several of the railings. At that time, FHWA determined that static loadings were insufficient to determine adequate railing performance and bridge railings were then required to be crash-tested to NCHRP 230 standards. AASHTO published its "Guide Specifications for Bridge Railings" in the late 80's, which began the use of performance level selection for given site conditions. In 1993, NCHRP 350 was published which included six different "Test Levels" for testing rails. Currently, the FHWA requires that all bridge railings installed on NHS projects meet the criteria of NCHRP 350 Test Level - 3 (TL-3).

4.4.2.1 TL-2 Rails

These rails have been tested to TL-2:

- Texas Classic Rail
- Fascia Mounted Thrie Beam
- TL-2 Timber Rail systems (refer to Figure 4-1 for examples)
- Any of the TL-4 or TL-5 options

4.4.2.2 TL-4 Rails

These rails have been tested to TL-4:

- Galvanized Steel Bridge Rail
- TL-4 Timber Rail system
- Maine Modified Kansas Rail
- F-Shaped Barrier (Type IIIA)

F-shaped barrier should be used if a TL-4 rail is required and the length of bridge rail is less than 35 feet. When the design length of bridge rail is 20

feet or less, the approach guardrail may be carried over the structure using a 3'-1 1/2" post spacing with a double layer of guardrail beam on the bridge structure and 50 feet beyond either end of the structure.

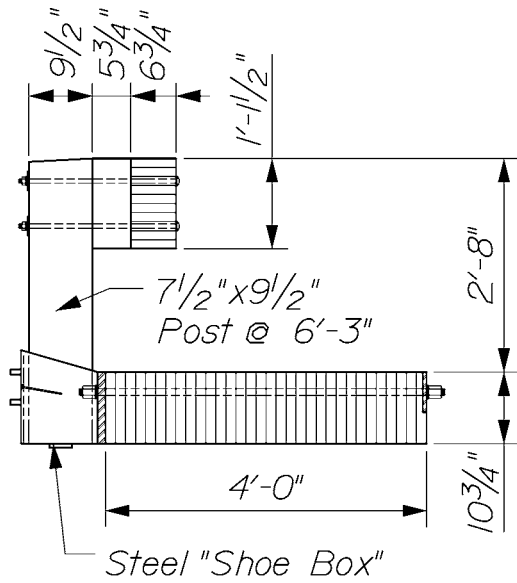
4.4.2.3 TL-5 Rails

This rail has been tested to TL-5: F-Shaped Barrier (Type IIIB)

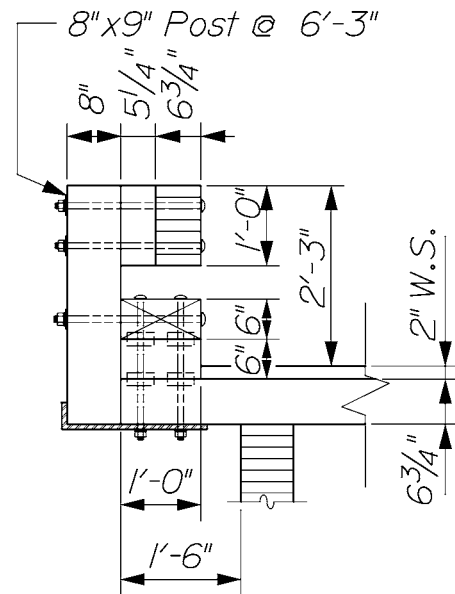
4.4.3 *Bridge Rail Selection*

The criteria in this subsection are meant for the selection of an appropriate bridge rail system for new construction projects only. For rehabilitation projects, refer to Section 10.5 Bridge Rail and Connections.

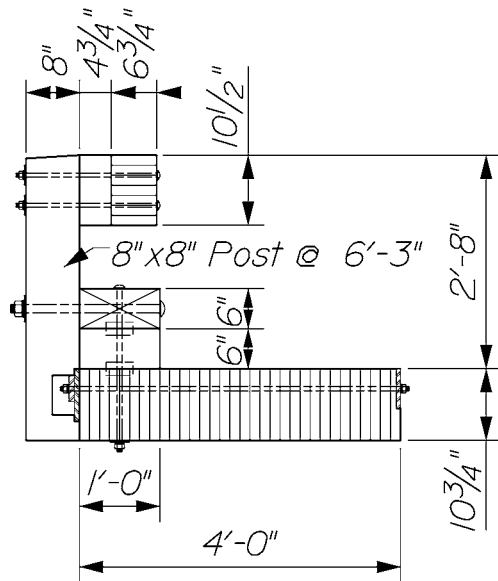
Regardless of the Adjusted ADT, the Structural Designer is expected to use good engineering judgment in the selection of a railing system. For example, it may be desirable to have a rigid railing system in an area where there is a retaining wall located immediately behind the guardrail, even if there is no bridge on the project.



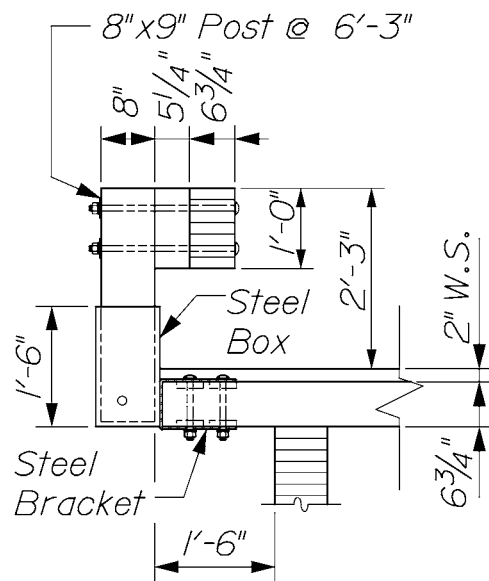
EXAMPLE NO. 1



EXAMPLE NO. 3



EXAMPLE NO. 2



EXAMPLE NO. 4

Figure 4-1 TL-2 Timber Rail Systems

Procedure 4-1 Rail Design

Step 1: Refer to Figure 4-2 and Figure 4-3 to determine K_c , K_g , & K_s .

Step 2: Calculate the adjusted ADT using the following equation:

$$\text{Adjusted ADT} = (\text{ADT}_{cy})(K_c)(K_g)(K_s)$$

Step 3: Look up the adjusted ADT in Table 4-6 for the specific design speed, truck percentage, and shoulder width.

Step 4: Compare the value from Table 4-6 with that calculated from the equation in step 2 above.

Step 5: Select test level based upon the following:

- If the value is less than the value in the TL-4 column, then a TL-2 rail is required, unless the project is on the NHS (refer to Figure 2-2) then a minimum of TL-3 is required.
- If the calculated Adjusted ADT is equal to or greater than the value from the TL-4 column but less than the value from the TL-5 column, then a TL-4 rail is required.
- If the value is equal to or greater than the value from the TL-5 column, then a TL-5 rail is the selected system.

Highway and railroad overpass structures should use an F-shaped barrier or a permanent snow fence attachment in the vicinity of the overpass in order to minimize snow and debris falling onto vehicles below. Should this barrier type be undesirable, the highway or railroad owner should be contacted to determine if an exception is acceptable.

Figure 4-2 K_g and K_c
Grade Traffic Adjustment Factor (K_g)
Curvature Traffic Adjustment Factor (K_c)

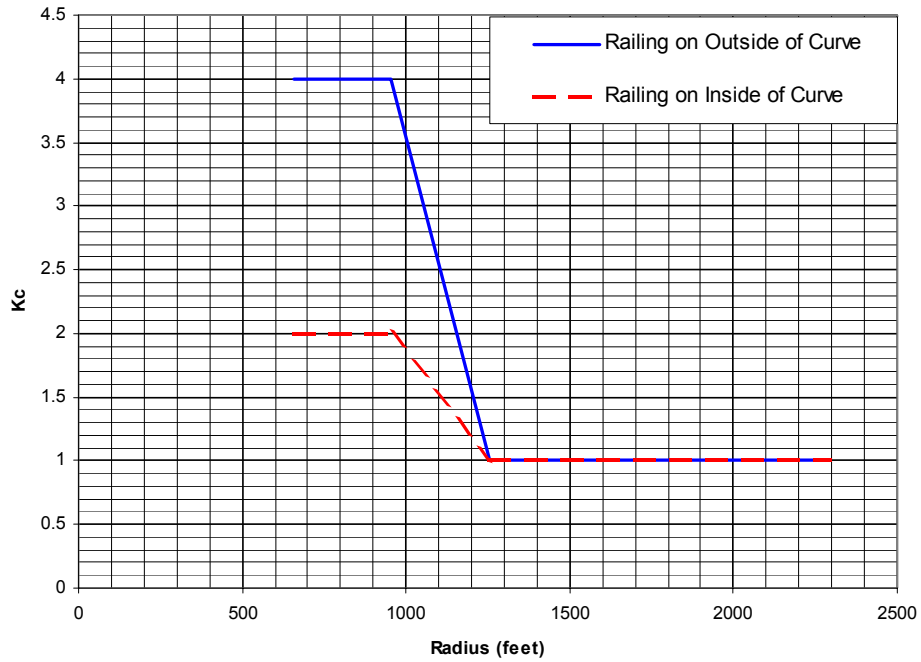
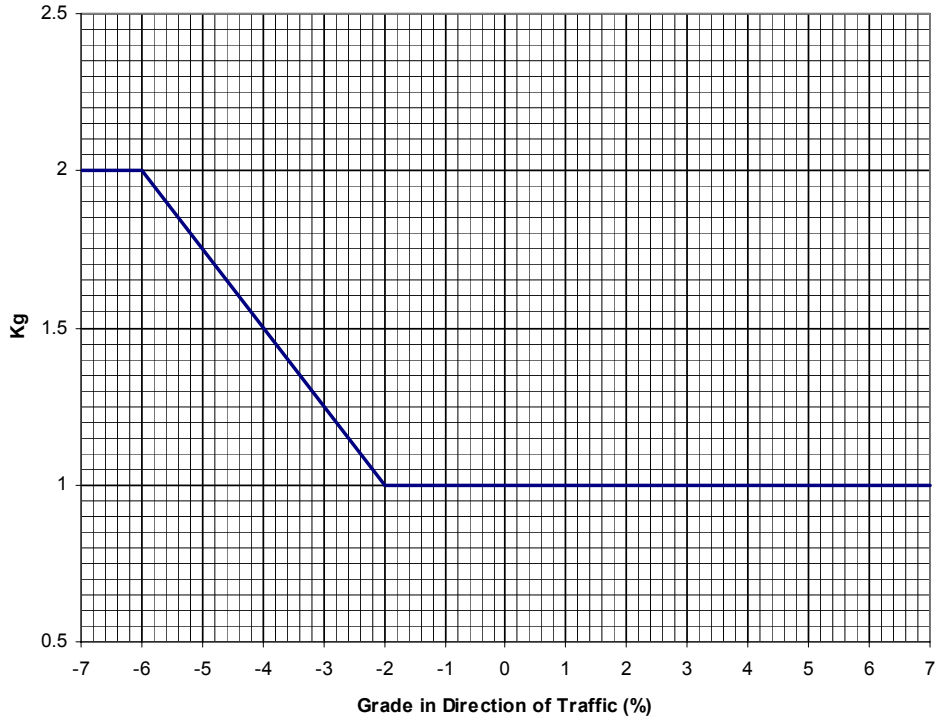


Figure 4-3 K_s
Traffic Adjustment Factor (K_s)
For Deck Height and Under Structure Conditions

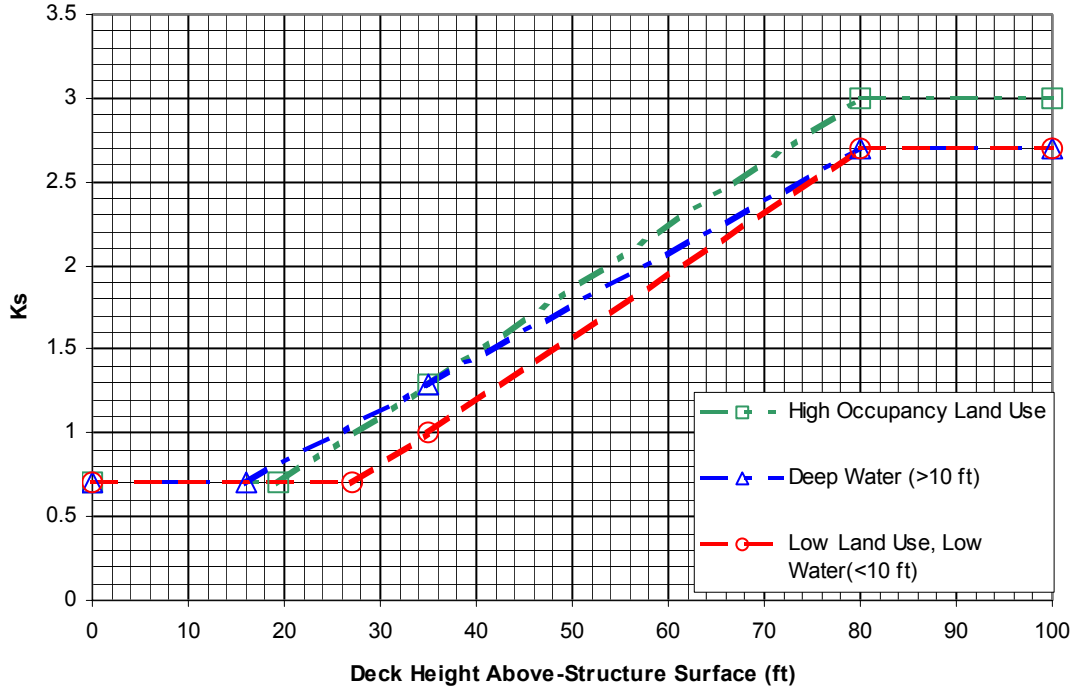


Table 4-6 Bridge Rail Performance Level Selection

Design Speed (mph)	Percent Trucks	Shoulder Width (ft)	Adjusted ADT for which a TL-4 or TL-5 is required					
			Divided or 5 + lanes		Undivided 4 lanes or less		One Way	
			TL-4	TL-5	TL-4	TL-5	TL-4	TL-5
30	0	0-3	151000	***	144300	***	75500	***
		3-7	283200	***	265200	***	141600	***
		7-12	***	***	***	***	316100	***
	5	0-3	56600	***	48000	***	28300	***
		3-7	90400	***	74600	***	45200	***
		7-12	148300	***	128900	***	74200	***
	10	0-3	23900	179800	19300	147900	12000	89900
		3-7	36500	258300	28800	228700	18300	129200
		7-12	55900	404400	46500	364600	28000	202200
	15	0-3	15100	102900	12100	84500	7600	51500
		3-7	22800	146600	17900	129200	11400	73300
		7-12	34400	228500	28300	205300	17200	114300
	20	0-3	11100	72000	8800	59100	5600	36000
		3-7	16600	102400	13000	90000	8300	51200
		7-12	24900	159200	20400	142900	12500	79600
40	0	0-3	19000	***	14400	***	9500	***
		3-7	24800	***	19000	***	12400	***
		7-12	33100	***	27200	***	16600	***
	5	0-3	14000	280700	10400	202400	7000	140400
		3-7	18000	335100	13400	253800	9000	167600
		7-12	24400	452000	19200	366700	12200	226000
	10	0-3	9800	79700	7100	55600	4900	39900
		3-7	12700	89800	9200	68600	6400	44900
		7-12	16900	132400	12800	102300	8500	66200
	15	0-3	7500	46400	5400	32200	3800	23200
		3-7	9800	51900	7000	39600	4900	26000
		7-12	12900	77600	9600	59400	6500	38800
	20	0-3	6100	32800	4400	22700	3100	16400
		3-7	8000	36500	5600	27900	4000	18300
		7-12	10400	54900	7700	41900	5200	27500
50	0	0-3	6200	***	4200	***	3100	***
		3-7	7200	***	5000	***	3600	***
		7-12	9900	***	7300	***	5000	***
	5	0-3	5500	162200	3700	107000	2800	81100
		3-7	6300	188600	4400	134100	3200	94300
		7-12	8400	247300	6100	171900	4200	123700
	10	0-3	4700	50000	3200	32000	2400	25000
		3-7	5400	61400	3700	41800	2700	30700
		7-12	7200	70600	5100	49300	3600	35300
	15	0-3	4100	29600	2800	18800	2100	14800
		3-7	4800	36700	3300	24800	2400	18400
		7-12	6300	41200	4400	28800	3200	20600
	20	0-3	3700	21000	2500	13300	1900	10500
		3-7	4300	26100	2900	17600	2200	13100
		7-12	5600	29100	3900	20300	2800	14600

Design Speed (mph)	Percent Trucks	Shoulder Width (ft)	Adjusted ADT for which a TL-4 or TL-5 is required					
			Divided or 5 + lanes		Undivided 4 lanes or less		One Way	
			TL-4	TL-5	TL-4	TL-5	TL-4	TL-5
60	0	0-3	3200	***	2000	***	1600	***
		3-7	3600	***	2300	***	1800	***
		7-12	4400	***	2900	***	2200	***
	5	0-3	3000	107300	1900	70300	1500	53700
		3-7	3300	126300	2100	82800	1700	63200
		7-12	4100	158400	2700	105600	2100	79200
	10	0-3	2800	39600	1800	25000	1400	19800
		3-7	3100	47500	2000	29300	1600	23800
		7-12	3900	53100	2500	33700	2000	26600
	15	0-3	2700	24300	1700	15200	1400	12200
		3-7	2900	29300	1900	17800	1500	14700
		7-12	3700	31900	2400	20000	1900	16000
	20	0-3	2500	17500	1600	10900	1300	8800
		3-7	2800	21100	1800	12800	1400	10600
		7-12	3500	22800	2200	14300	1800	11400
70	0	0-3	2200	191400	1300	165000	1100	95700
		3-7	2400	379100	1500	301500	1200	189600
		7-12	2800	***	1700	402400	1400	256400
	5	0-3	2100	63100	1300	42200	1100	31600
		3-7	2300	80000	1400	51600	1200	40000
		7-12	2700	96400	1600	64000	1400	48200
	10	0-3	2000	32100	1200	20000	1000	16100
		3-7	2300	38500	1400	22900	1200	19300
		7-12	2600	42200	1600	26700	1300	21100
	15	0-3	2000	21500	1200	13100	1000	10800
		3-7	2200	25300	1300	14700	1100	12700
		7-12	2600	27000	1600	16900	1300	13500
	20	0-3	1900	16200	1200	9700	1000	8100
		3-7	2100	18900	1300	10800	1100	9500
		7-12	2500	19900	1500	12300	1300	10000

4.4.4 Bicycle Railing

Bicycle bridge rail should be used on any bridge over 20 feet long where there is an established bicycle trail system or where high volumes of bicycle traffic are expected, as determined by the MaineDOT Bicycle and Pedestrian Coordinator.

The standard height for bicycle bridge rail is 42 inches.

4.4.5 Reduced Standard Bridge Rail

If the bridge is not on the NHS (refer to Figure 2-2), and the adjusted ADT is less than or equal to half of the maximum allowed for a TL-2 system, a rail

may be designed rather than crash-tested. The system may be designed in accordance with AASHTO LRFD Section 13 Appendix A for the TL-2 test condition. The railing must also meet all the geometric requirements for its proposed application found in AASHTO LRFD Section 13.

Consult with the bridge rail technical resource people for examples of recently designed bridge rails.

4.4.6 Aesthetics

Unfortunately, many of the crash-tested rails are often not considered to be aesthetically pleasing. If a TL-2 rail is appropriate, the Texas Classic Rail may be used when aesthetics is a concern. Consideration should also be given to color-galvanizing steel bridge rail to enhance its appearance. The required specification has been developed, along with specific color recommendations. For bridges satisfying the reduced standard criteria in Section 4.4.5, the Structural Designer may design an alternative attractive rail.

4.4.7 Transitions

For projects on the NHS, transitions from approach rail to bridge rail are required to meet the crash-testing conditions of NCHRP Report 350. The current standard details for transitions are based on the Alaskan Transition, which is 350 approved, with some minor modifications suggested by FHWA.

Use the following guidelines for transitions:

- o *Steel Bridge Rail:* For transitions on the NHS, use the Concrete Transition Barrier and the Bridge Transition Type 1. The bridge connections and approach guardrail transitions to the standard 2-bar steel bridge rail for bridges not on the NHS may consist of either the Steel Approach Railing or the Concrete Transition Barrier with the Bridge Transition Type 1.
- o *Timber Rail:* For approach guardrail transitions and bridge connections to a timber bridge rail, use a shoe attachment with doubled guardrail beam and 3'-1 1/2" post spacing. Either weathering steel or galvanized steel guardrail may be used. If steel backed timber guardrail is used, then the steel should be securely attached to the timber bridge rail.
- o *F-Shaped Barrier:* The approach guardrail should be stiffened and rigidly connected to the ends of the standard F-shaped concrete barrier with a Bridge Transition Type 1.
- o For one-way bridges on the NHS, the trailing end of the bridge rail need only be connected to the barrier ends with a 6'-3" post spacing and no doubled guardrail beam.
- o Regardless of the type of bridge rail selected, if the rail is pedestrian height, then it may be appropriate to have a pedestrian height railing or fence behind all or a portion of the approach railing, depending on site-specific conditions. (i.e., steepness of embankment, height of return wings, etc.)

4.5 Security Fences

The primary purpose of security fencing is to provide for the safety and security of pedestrians, and to prevent objects from being thrown or dropped from bridges to lower roadways, railroads, boat lanes, or occupied property. Certain overpass structures may warrant protective chain link fencing. Refer to the latest version of "A Guide for Protective Screening of Overpass Structures" for more information.

Adding a fence to a bridge structure should not be done routinely. It will increase maintenance responsibilities, and may exacerbate an existing sight distance problem. If a fence is used, it should be no higher than 6 feet to avoid limiting inspections with the under-bridge crane.

4.6 Wearing Surfaces

4.6.1 General

All bridges should have a 3 inch bituminous wearing surface plus a membrane except as follows:

- Bridges on local and collector roads with simple spans and an AADT less than 1000 should use a 1 inch integral concrete wearing surface.
- Bridges with an AADT over 1000 with grades in excess of 7%, or bridges where higher than usual braking or acceleration forces can be expected, such as at stop signs, exit and entrance ramps, or sharp horizontal curves should use a 2 inch unreinforced structural concrete wearing surface or a modified asphalt wearing surface system.

4.6.2 Descriptions

The types of wearing surfaces are described below:

4.6.2.1 Bituminous Wearing Surface with Membrane

The wearing surface consists of an impervious waterproofing membrane (nominally 1/4" thick) and approximately 3 inches of bituminous pavement of the grades specified on the plans, and placed in layers of the thickness shown in the Specifications.

4.6.2.2 Unreinforced Structural Concrete Wearing Surface

The wearing surface consists of an unreinforced structural concrete wearing surface with a thickness of 2 inches. The concrete used for the wearing surface is Class LP. The structural concrete wearing surface should be treated with protective coating for concrete surfaces.

4.6.2.3 Integral Concrete Wearing Surface

The wearing surface consists of an extra 1 inch cover over the top of the deck reinforcement for a total concrete cover of 3 inches. The extra inch of concrete should be included in the computations as dead load, but should be excluded from the slab section capacity computations. No allowance is made in the computations for future overlays or wearing surfaces. The concrete used for the slab and wearing surface is Class A. The integral

concrete wearing surface should be treated with protective coating for concrete surfaces.

4.6.2.4 Modified Asphalt Wearing Surface System

The wearing surface consists of approximately 3 inches of impervious hot mix modified asphalt placed on a high performance membrane.

Rosphalt, a proprietary product, is one type of modified asphalt system that does not require a separate membrane. The use of this product is limited to wearing surface replacement projects - concrete wearing surfaces, in particular. Rosphalt may only be used when the proprietary item approval process has been followed.

Prior to calling for a modified asphalt wearing surface system on a project, the Designer should consult with a pavement technical resource person for guidance in the appropriate system to use.

4.7 Membranes

High performance waterproofing membrane should be used under bituminous wearing surfaces on most bridge structures. The prequalified list of standard and high performance waterproofing membrane systems can be found on the MaineDOT website at: <http://www.maine.gov/mdot/transportation-research/approved-products/waterproof-membrane-systems.php>.

Standard membrane should be used on concrete buried structures, placed directly on top of the concrete and wrapped down one foot along the vertical wall.

4.8 Deck Joints and Expansion Devices

4.8.1 General

Deck joints add cost to the structure, increase maintenance requirements, and should be avoided whenever possible. Integral abutments should be used (refer to Section 5.4.2, Integral Abutments) or the slab should be carried over the backwall (refer to Section 6.2.2 Decks) whenever possible. The Designer must become familiar with the Standard Details (520 and 521), as well as applicable manufacturer's product information, before specifying an expansion device for a particular project.

In all other cases, deck joints with appropriate expansion devices will be necessary. The choice of which expansion device to use depends upon the

movement rating, which is the magnitude of expected expansion and contraction of the structure due to temperature change. The movement rating is the maximum movement from extreme cold to extreme hot, and is calculated as 1-1/4" per 100 feet of bridge expansion length from a fixed bearing. Compression seals are used for a movement rating up to 2-1/2". Gland seals are used for a movement rating up to 3 inches. Finger joints are used up to about 12 inches. Extrapolation of finger joint dimensions or modular joints may be used for larger movement ratings.

Commentary: The expansion rate of 1-1/4" per 100 feet of bridge is based upon the coefficient of expansion for steel. The rate may be used for the determination of the movement rating on all bridge structures either steel or concrete. If a more precise determination of the movement rating for a concrete structure is required, the movement rating may be calculated using the coefficient of expansion for concrete from AASHTO LRFD.

For movement ratings approaching 2-1/2", either a compression seal or gland seal may be used. Whether or not a gland seal can be used will depend upon the minimum opening supplied by the manufacturer.

Special design consideration is required for skews between 30° and 50° back on the right (skewed either way on the Interstate) because of the hazard of a snowplow blade catching in the joint.

4.8.2 *Preformed Elastomeric Joint Seals*

Preformed Elastomeric Joint Seals (Compression Seals) should be specified on the plans in accordance with the Standard Details 520 (08-14) and Appendix D Standard Notes Superstructures.

The Designer will calculate the movement rating, and then specify the expansion device based upon that rating to the nearest 1/8". At fixed bearings that require a deck joint (i.e. non-slab over backwall), a movement rating of 1/2" should be specified, unless an engineering evaluation of the joint geometry indicates the need for a larger value. The maximum opening of any joint is limited to 3-1/2" in the direction of the centerline of the roadway. The Designer should verify that the opening associated with the specified movement rating would not exceed the seal size. Refer to Example 4-1.

Listed in Table 4-7 are the compression seals prequalified for the movement ratings indicated.

*Commentary: Table 4-7 was developed based on pressure-deflection tests performed by the University of Maine on samples furnished by the manufacturers. The tested samples were also evaluated for their ability to absorb racking movement. The skews shown in the table are based on that evaluation. This table may also be found at the MaineDOT product approval web page at the following web address:
<http://www.maine.gov/mdot/transportation-research/approved-products/compression-seals.php>*

Table 4-7 Elastomeric Joint Seal Movement Ratings

Movement Rating (in)	Maximum Expansion Length (ft)	Manufacturer			
		D.S. Brown Company		Watson Bowman Acme Corporation	
		Seal	Maximum Skew	Seal	Maximum Skew
0.500	40	CV1625	40°	WA175	35°
0.625	50	CV1752	35°	WA175	35°
0.750	60	CV2000* or H2001*	35°	WA200	35°
0.875	70	CV2502 or H2505	30°	WA250	30°
1.000	80	CV2502 or H2505	30°	WA250	30°
1.125	90	CV2502 or H2505	30°	WA300	25°
1.250	100	CV3000 or H3000	25°	WA300	25°
1.375	110	CV3000 or H3000	25°	WA350	25°
1.500	120	CV3000 or H3000	25°	WA350	25°
1.625	130	CV3500 or H3500	25°	WA400	25°
1.750	140	CV3500 or H3500	25°	WA500	25°
1.875	150	CV4000 or H4000	25°	WA500	25°
2.000	160	CV4000 or H4000	25°	WA500	25°
2.250	180	CA5001 or H5000	25°	WA500	25°
2.500	200	CA5001 or H5000	25°		

*Use for retrofitting of existing joints only.

Note: The movement rating shown is to be used as the actual movement of the deck joint parallel to the direction of expansion. Although the Standard Details show a movement rating at 45°F to be normal to the centerline of bearings, no reduction in the movement rating will be made for skewed structures. This will compensate for movements in the transverse direction.

The Contractor sets the opening in the field based upon the formula given in Standard Detail 520 (14). The two applicable seals are determined from Table 4-7 for the design movement rating. Refer to Example 4-1 for guidance.

Example 4-1 Compression Seal Design

The calculated movement rating is 1.75 inches. The nominal width is found in the manufacturer's designation, e.g. CV3500 is a 3-1/2" seal, and WA500 is a 5 inch seal. This width is based upon the relaxed dimension of the seal. The 0.85 multiplier is used to assure the seal is always in compression. The WA500 from Watson Bowman is tried to determine its applicability to the deck joint.

Step 1: Find the opening at 45°F, X.

$$X = (0.85 \times \text{nominal width}) - (1/2 \times \text{movement rating})$$

$$X = (0.85 \times 5) - 1/2 (1.75) = 3.375" \text{ at } 45^\circ\text{F} \text{ (refer to Note 4, Standard Detail 520 (14))}$$

Step 2: Find the maximum opening

$$\text{Max opening} = 3.375 + 1/2(1.75) = 4.25" \text{ at } -30^\circ\text{F}$$

Since this exceeds the maximum allowable opening of 3-1/2", a different manufacturer's seal should be used or a special sliding plate configuration will be necessary. Therefore, try the DS Brown seal.

Step 3: Repeat previous steps for another seal.

$$X = (0.85 \times 3.5) - 1/2 (1.75) = 2.10" \text{ at } 45^\circ\text{F}$$

$$\text{Maximum opening} = 2.10 + 1/2(1.75) = 2.98" \text{ at } -30^\circ\text{F}$$

This is less than 3-1/2". Thus, a compression seal may be used, either with or without sliding plates. Far more frequently the Contractor elects to avoid the sliding plates configuration and uses the compression seal.

4.8.3 Gland Seals

Gland Seals should be specified on the plans in accordance with Standard Details 520 (01-07) and Appendix D Standard Notes Superstructures.

The 4 inch nominal gland seal is the only size that is allowed by MaineDOT. The minimum opening for a gland seal is set at 1/2". This differs from the manufacturer's recommendation of 0 inches minimum. Also, the maximum allowable opening allowed by MaineDOT is set at 3-1/2". Therefore, when the Designer uses a 4 inch nominal gland seal, the actual maximum movement rating allowed will be 3 inches (3-1/2" minus 1/2").

The maximum allowed skew for a gland seal is 45°. This is limited by the racking ability of the seal.

4.8.4 Finger Joints

Finger joints should be specified on the plans in accordance with Standard Details 521(01-11) and Appendix D Standard Notes Superstructures. Standard Detail 521(10) provides a table that specifies dimensions required for a given span length and skew.

This is the expansion device most commonly used for bridges with a movement rating of greater than 3 inches. If bicycle and/or pedestrian traffic is a major concern, consideration should be given to placing a sliding plate on top where needed to cover the opening.

To collect the roadway drainage through the finger joint, a curtain or trough system must be used. The curtain system (Figure 4-4) protects the girder by placing neoprene curtains at the end of each girder, and dumping the water diffusely on a protected bridge seat, which is periodically cleaned by Bridge Maintenance. The curtain system is usually preferred because of the easy access for maintenance, and its effectiveness in protecting the girder by eliminating drains which may plug.

The trough system (Figure 4-5) collects the water and directs it via a drain to a specific location. Debris that collects in troughs should be cleaned out by Bridge Maintenance before it accumulates to the point where it is too heavy for the trough to support. Although the curtain system is preferred, the trough system may be necessary if adjacent buildings or other site features must be protected from diffuse spray.

4.8.5 Modular Joints

Modular joints are usually quite expensive compared to other expansion devices, and have had a poor record in performance and maintenance. They should be used only under special circumstances with permission from the Engineer of Design, such as when using a curtain or trough system becomes problematic, and the additional cost can be justified. Modular joints should be specified on the plans in accordance with Standard Specifications Section 522 – Expansion Devices - Modular. Only gland type seals are permitted. The Designer will provide the anticipated movement rating to the manufacturer, who will design the modular joint. The model selection should be based on data from Watson-Bowman-Acme and/or D. S. Brown.

The dimensions for the block-out must be determined, shown on the design drawings, and verified by the manufacturer. It is suggested that 1'-6" be used for movements up to and including 10 inches. For movements larger than 10 inches, the block-out width and the backwall width must be increased by 4 inches for each additional 3 inches of movement or fraction thereof. The dimension of 1'-3" between the face of backwall and centerline of bearings can be used for movements up to and including 6 inches. For movements larger than 6 inches this dimension must be increased 3 inches for each additional 3 inches of movement or fraction thereof.

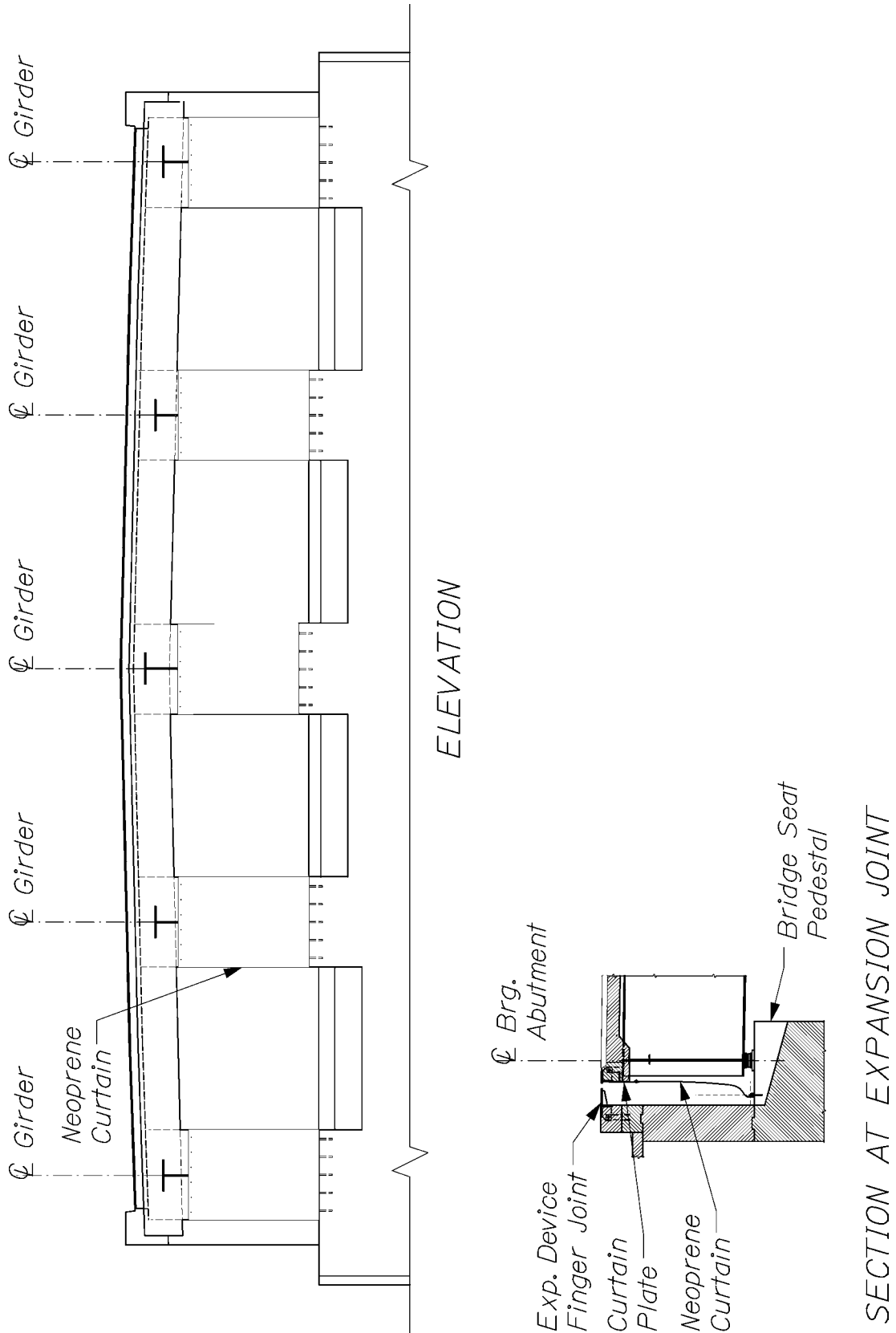


Figure 4-4 Curtain System

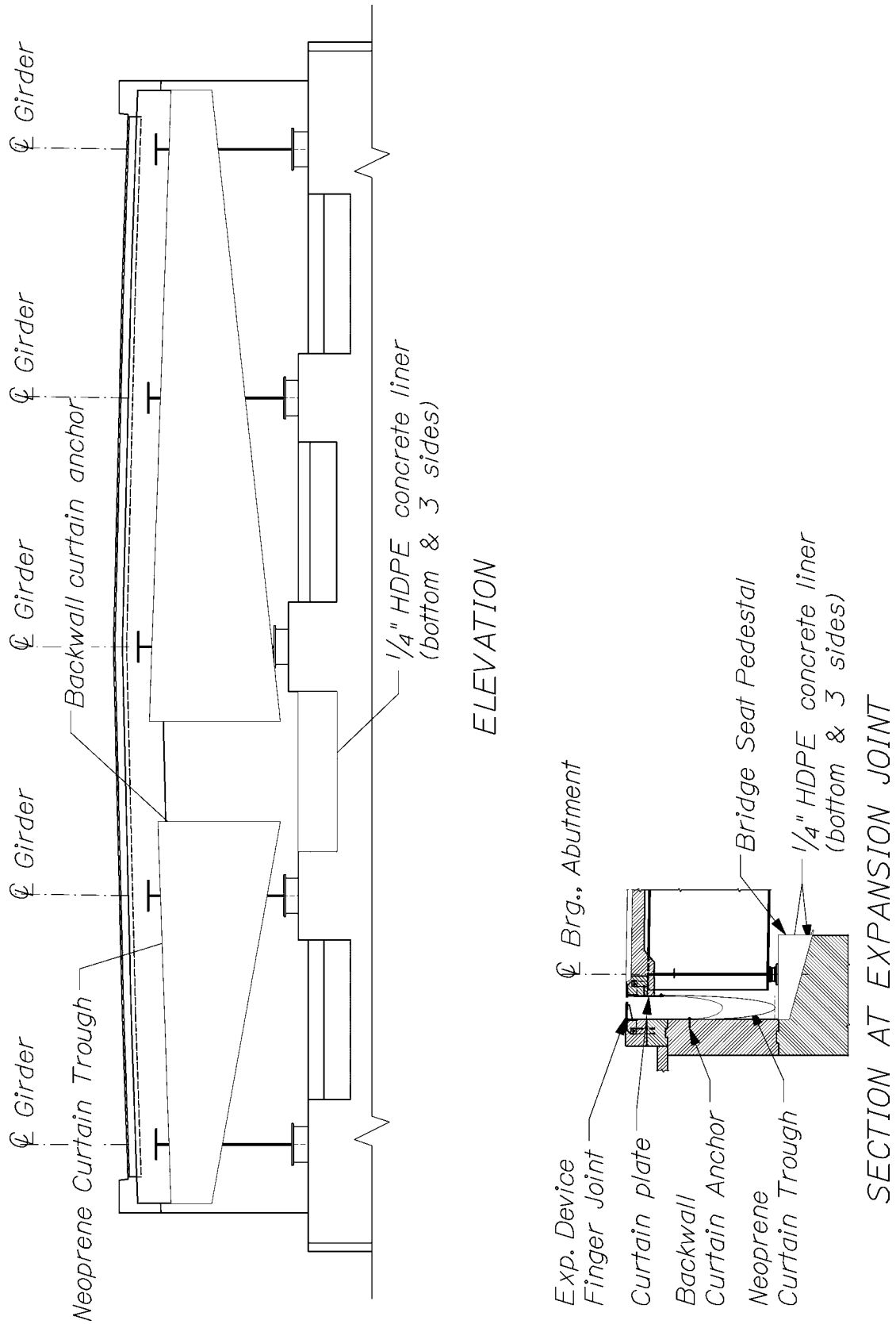


Figure 4-5 Trough System

4.8.6 Design/Contract Drawings Presentation

The selected expansion device (compression seal, gland seal, finger joint, etc.) should be shown in the body of the drawing, commonly on the superstructure slab view, as well as included in the estimate sheet.

Movement ratings for all expansion devices should be shown on the drawing. A design drawing with a compression seal should also show a temperature adjustment chart. Other expansion device adjustments are calculated in the field through a formula in the Standard Details, and need not be shown on the plans.

Special attention should be paid to unique deck joint situations, such as finger joints beyond the standard detail largest size, modular joints, and gland seal joints that have the potential of exceeding the 3-1/2" opening limit.

4.9 Drains

4.9.1 Design

When required, bridge drains should be used as detailed in Standard Details 502. All modifications to the Standard Details must be approved by the Engineer of Design. If a custom drain is needed, the following criteria should be met:

- o The minimum width dimension for the downspout should be 8 inches whenever possible, to help avoid clogging.
- o The opening for the drain should be kept out of the wheel path.
- o The grating should be bicycle-friendly (perpendicular to traffic).

Bridge drain spacing should begin at a convenient spot on the bridge, i.e. the low end, and progress across the structure. Drains should be placed so that splash onto substructure units does not occur. Preferably, drains should not be placed within 10 feet of a pier or abutment. For overpasses, drains should not be placed over the roadway or the railroad tracks. At the vertex of a sag vertical curve, provide one extra drain on either side of the drain at the low point.

When steep grades are involved, consideration should be given to closely spacing two drains at the low end of a bridge to minimize runoff erosion at the abutment from drain overflow. Approaches to the bridge should be designed to handle runoff from the bridge.

At the ends of concrete barriers, wingwalls, and at other locations where roadway runoff is likely to concentrate, erosion protection should be provided on side slopes by the use of sod or riprap down spouts, catch basins and outlet culverts, or other approved means of erosion control. Concrete splash blocks should be used under the discharge of deck drains on slopes not otherwise protected by either riprap or slope pavement.

4.9.2 Use of Standard Details

Standard Details Section 502 has three standard types of bridge drains. Drain Type A has a 2'-9" long by 1 foot wide grate with an 8 inch by 12 inch downspout. The length of the grate is based on a 3 foot over hang with a 12 inch or 16 inch wide beam flange. The length of the grate may need to be increased for beams with wider flanges or larger overhangs.

Drain Type B consists of a 1-4" by 1 foot grate with a 1 foot by 1 foot downspout. This drain should be used on bridges with narrow shoulders. Typically, these drains are used on bridges with a curb-to-curb width of 28 feet or less with shoulders 2 feet wide or narrower. Assuming a 12 inch wide beam flange, the maximum overhang dimension is 1'-11".

Drain Type C consists of a 1 foot by 1 foot grate with and a straight 1 foot by 1 foot downspout. This drain is the least prone to clogging. Typically, this drain would be used next to sidewalks or on bridges with somewhat large overhangs. Assuming a 12 inch wide flange and a 1'-8" wide curb, the minimum overhang dimension is 3'-7". One disadvantage with this drain is aesthetics, since the drain is on the outside of the beam and not hidden from view.

A standard drain has not been developed for butted precast concrete voided slabs and box beams. The preferred drainage system for this structure type is no drains at all. To avoid drains, the shoulder must be wide enough or the bridge length short enough to preclude the need for drains. If drains are needed, an opening in the curb through to the fascia is required. Then the fascia must be protected from the salt-laden water.

4.9.3 Bridge Drain Spacing Tables

The maximum bridge drain spacing can be determined using Table 4-8 through Table 4-14 for drain Type A and Table 4-15 through Table 4-28 for drain Types B and C. The maximum drain spacing is 300 feet. Consider the following exceptions when using the tables:

- o Bridges over 200 feet in length will require at least one drain per drainage area located at the low end.

- o If a vertical curve is on the bridge, the tables cannot be used.
- o Consideration should be given to reducing the bridge drain spacing from what the tables allow for bridges with drainage lengths over 200 feet. Long flow lengths with few drains should be avoided due to potential hazards if some drains become clogged.

The information needed for the proper use of the tables includes:

- o Cross-slope of the bridge and whether or not the elevated shoulder of a superelevated section is included in the drainage area
- o Grade of the bridge
- o Number of lanes to be drained
- o Shoulder width
- o Bridge length

Commentary: The procedure used to develop these tables is referenced in FHWA HEC 12 and FHWA HEC 21. A Q10 storm frequency was assumed with a rainfall intensity of 5 inches per hour. The drain spacing tables are based on the assumption that the bridge has a constant grade. If a vertical curve is on the bridge, the tables cannot be used. The Designer should refer to HEC 21 for the design procedure for a bridge on a vertical curve.

Table 4-8 Bridge Drain Type A
Maximum Bridge Drain Spacing
Normal Crown Deck
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	1	20	20	45	45	88	85	83	80	142	127	208	171
	2	13	13	31	31	55	55	53	53	85	76	130	107
1	1	22	22	62	62	125	120	117	113	201	179	295	240
	2	13	13	34	34	55	52	69	66	121	107	184	150
1.5	1	27	27	76	76	153	146	144	138	246	218	361	293
	2	15	15	42	42	87	84	84	81	148	131	226	183
2	1	32	32	87	87	176	169	166	159	285	252	417	338
	2	17	17	48	48	101	96	97	93	171	151	260	211
2.5	1	35	35	98	97	197	189	186	178	318	281	466	377
	2	19	19	54	54	113	108	109	104	191	169	291	236
3	1	39	39	107	107	216	207	203	195	348	308	510	413
	2	21	21	59	59	123	118	119	114	209	185	319	258
3.5	1	42	42	115	115	233	223	220	210	376	333	551	439
	2	23	23	64	64	133	128	129	123	226	200	345	275
4	1	45	45	123	123	250	239	235	225	402	356	589	460
	2	24	24	69	69	143	136	138	132	241	213	368	287
4.5	1	48	48	131	131	265	253	249	238	427	371	625	478
	2	26	26	73	73	151	145	146	140	256	223	391	299
5	1	50	50	138	138	279	267	263	251	450	385	659	495
	2	27	27	77	77	159	152	154	147	270	231	412	309

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-9 Bridge Drain Type A
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	13	13	31	31	55	55	53	53	85	76	130	107
	4	9	9	20	20	36	36	36	36	56	56	81	81
1	2	13	13	34	34	71	68	69	66	121	107	184	150
	4	9	9	20	20	38	37	38	36	67	60	105	86
1.5	2	15	15	42	42	87	84	84	81	148	131	226	183
	4	9	9	22	22	47	45	46	44	82	73	129	105
2	2	17	17	48	48	101	96	97	93	171	151	260	211
	4	9	9	26	26	54	52	53	51	95	84	149	121
2.5	2	19	19	54	54	113	108	109	104	191	169	291	236
	4	10	10	29	29	61	58	60	57	106	94	166	135
3	2	21	21	59	59	123	118	119	114	209	185	319	258
	4	11	11	31	31	66	64	65	62	116	103	182	147
3.5	2	23	23	64	64	133	128	129	123	226	200	345	275
	4	12	12	34	34	72	69	70	67	125	111	197	157
4	2	24	24	69	69	143	136	138	132	241	213	368	287
	4	13	13	36	36	77	73	75	72	134	119	210	164
4.5	2	26	26	73	73	151	145	146	140	256	223	391	299
	4	13	13	39	38	81	78	80	76	142	124	223	171
5	2	27	27	77	77	159	152	154	147	270	231	412	309
	4	14	14	41	41	86	82	84	81	150	128	235	177

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-10 Bridge Drain Type A
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	13	13	31	31	55	55	53	53	84	84	118	118
	4	9	9	20	20	36	36	36	36	56	56	81	81
1	2	13	13	31	31	62	60	59	56	101	89	147	120
	4	9	9	20	20	36	36	36	36	60	56	92	81
1.5	2	14	14	38	38	76	73	72	69	123	109	180	147
	4	9	9	21	21	44	42	42	40	74	66	113	92
2	2	16	16	44	44	88	84	83	79	142	126	208	169
	4	9	9	24	24	50	48	49	47	85	76	130	106
2.5	2	18	18	49	49	99	94	93	89	159	141	233	189
	4	10	10	27	27	56	54	54	52	95	84	146	118
3	2	19	19	53	53	108	103	102	97	174	154	255	206
	4	10	10	30	30	62	59	60	57	105	92	159	129
3.5	2	21	21	58	58	117	112	110	105	188	166	276	220
	4	11	11	32	32	67	64	64	62	113	100	172	137
4	2	22	22	62	62	125	119	117	112	201	178	295	230
	4	12	12	34	34	71	68	69	66	121	107	184	144
4.5	2	24	24	65	65	132	127	125	119	213	186	313	239
	4	13	13	36	36	76	72	73	70	128	111	195	150
5	2	25	25	69	69	139	133	131	126	225	192	329	247
	4	13	13	38	38	80	76	77	74	135	115	206	155

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-11 Bridge Drain Type A
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
4% Cross Slope (0.48"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	36	36	83	83	160	154	155	149	272	242	414	340
	4	23	23	54	54	98	98	97	97	151	135	237	194
1	2	38	38	109	109	227	217	219	210	384	341	586	477
	4	23	23	58	58	122	117	120	115	213	189	335	273
1.5	2	47	47	134	133	278	266	268	257	470	417	718	567
	4	24	24	71	71	150	143	147	141	261	232	410	324
2	2	54	54	154	154	321	307	310	296	543	465	829	623
	4	28	28	82	82	173	165	169	162	302	258	474	356
2.5	2	61	61	173	172	359	336	346	324	607	500	927	666
	4	32	32	91	91	193	181	189	177	337	278	529	380
3	2	66	66	189	189	393	357	379	345	665	529	1015	699
	4	35	35	100	100	212	192	208	189	370	294	580	399
3.5	2	72	72	204	201	424	375	410	362	719	552	1096	724
	4	37	37	108	106	229	202	224	198	399	307	626	414
4	2	77	77	218	210	454	391	438	377	768	571	1172	745
	4	40	40	116	111	244	210	240	206	427	317	670	425
4.5	2	81	81	231	219	481	404	465	390	815	587	1243	760
	4	42	42	123	116	259	218	254	214	453	326	710	434
5	2	86	86	244	227	507	416	490	401	859	600	1310	772
	4	45	45	129	120	273	224	268	220	477	333	749	441

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-12 Bridge Drain Type A
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
4% Cross Slope (0.48"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	36	36	83	83	148	148	145	145	227	227	331	272
	4	23	23	54	54	98	98	97	97	153	153	219	219
1	2	36	36	98	98	198	190	187	179	320	284	469	382
	4	23	23	55	54	113	109	110	105	192	171	293	238
1.5	2	44	44	120	120	243	233	229	219	392	347	574	454
	4	24	24	67	67	139	133	134	128	235	208	359	283
2	2	50	50	139	139	281	269	264	253	453	387	663	498
	4	27	27	77	77	160	153	155	148	272	232	414	311
2.5	2	56	56	155	155	314	294	295	276	506	417	741	532
	4	30	30	86	86	179	168	173	162	304	250	463	333
3	2	62	62	170	170	344	312	324	294	554	441	812	559
	4	33	33	95	94	196	179	190	172	333	264	507	349
3.5	2	67	67	184	181	371	328	349	309	599	460	877	580
	4	36	36	102	100	212	188	205	181	359	276	548	362
4	2	71	71	196	189	397	342	374	322	640	476	938	596
	4	38	38	109	105	227	195	219	189	384	286	586	372
4.5	2	76	76	208	197	421	354	396	333	679	489	994	608
	4	41	41	116	109	241	202	232	195	407	294	622	380
5	2	80	80	220	204	444	364	418	342	716	500	1048	617
	4	43	43	122	113	254	208	245	201	430	300	655	386

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-13 Bridge Drain Type A
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
6% Cross Slope (0.72"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	65	65	152	152	316	303	305	293	535	477	816	668
	4	42	42	97	97	176	176	173	173	297	265	466	382
1	2	76	76	215	215	446	428	431	413	756	663	1153	894
	4	42	42	114	114	240	230	236	226	420	369	659	511
1.5	2	93	93	263	263	547	510	528	492	926	759	1413	1010
	4	48	48	139	139	294	274	289	269	514	422	807	577
2	2	107	107	304	300	631	560	610	541	1069	826	1631	1085
	4	56	56	161	159	340	302	334	296	594	459	932	620
2.5	2	119	119	340	324	706	599	682	578	1196	873	1824	1134
	4	62	62	180	171	380	323	373	317	664	485	1042	648
3	2	131	131	372	343	773	629	747	607	1310	907	1998	1164
	4	68	68	197	182	416	339	409	332	728	504	1142	665
3.5	2	141	139	402	360	835	653	806	630	1415	930	2158	1180
	4	73	72	213	190	450	351	441	345	786	517	1233	674
4	2	151	146	430	374	893	671	862	648	1512	946	2307	1184
	4	79	76	227	198	481	361	472	354	840	525	1318	677
4.5	2	160	152	456	385	947	685	914	661	1604	955	2447	1179
	4	83	79	241	204	510	369	500	362	891	530	1398	674
5	2	169	157	480	395	998	695	964	671	1691	958	2579	1167
	4	88	82	254	209	538	374	527	367	939	532	1474	667

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-14 Bridge Drain Type A
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
6% Cross Slope (0.72"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	65	65	149	149	276	265	260	250	446	397	652	535
	4	42	42	97	97	176	176	173	173	274	274	408	334
1	2	70	70	193	193	391	374	368	352	630	553	923	715
	4	42	42	107	107	223	214	216	206	378	332	577	447
1.5	2	86	86	237	236	478	446	450	420	772	633	1130	808
	4	46	46	132	131	273	255	264	246	463	380	706	505
2	2	99	99	273	270	552	490	520	462	891	688	1305	868
	4	53	53	152	150	316	280	305	271	535	413	816	543
2.5	2	111	111	306	291	618	524	581	493	996	727	1459	907
	4	60	60	170	162	353	300	341	289	598	436	912	567
3	2	122	122	335	309	677	551	637	518	1091	756	1598	931
	4	65	65	186	172	387	315	373	304	655	453	999	582
3.5	2	131	129	362	324	731	571	688	537	1179	775	1726	944
	4	71	70	201	180	418	326	403	315	707	465	1079	590
4	2	140	135	387	336	781	587	735	553	1260	788	1845	947
	4	76	73	215	187	446	335	431	324	756	473	1153	592
4.5	2	149	141	410	347	829	599	780	564	1337	796	1957	944
	4	80	76	228	193	474	342	457	331	802	477	1223	590
5	2	157	146	432	355	874	609	822	573	1409	798	2063	934
	4	84	79	240	197	499	348	482	336	845	479	1290	584

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

**Table 4-15 Bridge Drain Type B
Maximum Bridge Drain Spacing
Normal Crown Deck
2% Cross Slope (0.24"/ft)**

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	1	20	20	45	45	88	62	83	59	142	87	208	111
	2	13	13	31	31	55	55	53	53	85	52	130	69
1	1	22	21	62	50	125	86	117	81	201	118	295	151
	2	13	13	34	31	71	49	69	47	121	71	184	94
1.5	1	27	26	76	61	153	104	144	98	246	143	361	182
	2	15	14	42	34	87	59	84	57	148	86	226	114
2	1	32	30	87	70	176	119	166	112	285	164	417	209
	2	17	16	48	39	101	68	97	66	171	98	260	131
2.5	1	35	34	98	78	197	133	186	125	318	183	466	233
	2	19	18	54	43	113	76	109	73	191	110	291	145
3	1	39	37	107	85	216	145	203	136	348	199	510	254
	2	21	20	59	47	123	83	119	80	209	120	319	159
3.5	1	42	40	115	92	233	156	220	147	376	215	551	270
	2	23	21	64	51	133	89	129	86	226	129	345	169
4	1	45	43	123	98	250	167	235	157	402	229	589	282
	2	24	23	69	55	143	95	138	92	241	138	368	176
4.5	1	48	45	131	104	265	177	249	166	427	239	625	293
	2	26	24	73	58	151	101	146	97	256	144	391	183
5	1	50	48	138	110	279	186	263	175	450	248	659	303
	2	27	26	77	61	159	106	154	103	270	149	412	189

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-16 Bridge Drain Type B
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	13	13	31	31	55	55	53	53	85	52	130	69
	4	9	9	20	20	36	36	36	36	47	29	74	40
1	2	13	13	34	31	71	55	69	53	121	71	184	94
	4	9	9	20	20	38	36	38	36	67	39	105	54
1.5	2	15	14	42	34	87	59	84	57	148	86	226	114
	4	9	9	22	20	47	36	46	36	82	48	129	65
2	2	17	16	48	39	101	68	97	66	171	98	260	131
	4	9	9	26	21	54	37	53	36	95	55	149	75
2.5	2	19	18	54	43	113	76	109	73	191	110	291	145
	4	10	9	29	23	61	41	60	40	106	61	166	83
3	2	21	20	59	47	123	83	119	80	209	120	319	159
	4	11	10	31	25	66	45	65	44	116	66	182	91
3.5	2	23	21	64	51	133	89	129	86	226	129	345	169
	4	12	11	34	27	72	48	70	47	125	72	197	96
4	2	24	23	69	55	143	95	138	92	241	138	368	176
	4	13	12	36	29	77	51	75	50	134	76	210	101
4.5	2	26	24	73	58	151	101	146	97	256	144	391	183
	4	13	13	39	31	81	54	80	53	142	80	223	105
5	2	27	26	77	61	159	106	154	103	270	149	412	189
	4	14	13	41	32	86	57	84	56	150	83	235	108

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-17 Bridge Drain Type B
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	13	13	31	31	55	55	53	53	84	84	118	118
	4	9	9	20	20	36	36	36	36	43	26	65	35
1	2	13	13	31	31	62	55	59	53	101	84	147	118
	4	9	9	20	20	36	36	36	36	60	36	92	47
1.5	2	14	13	38	30	76	52	72	49	123	84	180	118
	4	9	9	21	20	44	30	42	36	74	43	113	57
2	2	16	15	44	35	88	60	83	56	142	84	208	118
	4	9	9	24	19	50	34	49	36	85	49	130	65
2.5	2	18	17	49	39	99	66	93	62	159	91	233	118
	4	10	9	27	22	56	38	54	37	95	55	146	73
3	2	19	18	53	43	108	72	102	68	174	100	255	127
	4	10	10	30	24	62	41	60	40	105	60	159	79
3.5	2	21	20	58	46	117	78	110	73	188	107	276	135
	4	11	11	32	26	67	45	64	43	113	64	172	84
4	2	22	21	62	49	125	83	117	78	201	115	295	141
	4	12	11	34	27	71	48	69	46	121	69	184	88
4.5	2	24	23	65	52	132	88	125	83	213	120	313	147
	4	13	12	36	29	76	50	73	49	128	72	195	92
5	2	25	24	69	55	139	93	131	88	225	124	329	151
	4	13	13	38	31	80	53	77	51	135	74	206	95

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-18 Bridge Drain Type B
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
4% Cross Slope (0.48"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	36	36	83	83	160	112	155	109	272	164	414	219
	4	23	23	54	54	98	98	97	97	151	91	237	125
1	2	38	37	109	88	227	155	219	150	384	225	586	299
	4	23	23	58	54	122	98	120	97	213	125	335	171
1.5	2	47	45	134	108	278	188	268	181	470	272	718	352
	4	24	23	71	57	150	101	147	99	261	151	410	201
2	2	54	52	154	124	321	216	310	208	543	302	829	385
	4	28	27	82	66	173	116	169	114	302	168	474	220
2.5	2	61	58	173	138	359	235	346	227	607	324	927	410
	4	32	30	91	73	193	127	189	124	337	180	529	234
3	2	66	63	189	151	393	250	379	241	665	342	1015	430
	4	35	33	100	80	212	135	208	132	370	190	580	246
3.5	2	72	68	204	160	424	262	410	253	719	357	1096	445
	4	37	35	108	85	229	141	224	139	399	198	626	254
4	2	77	73	218	168	454	273	438	264	768	369	1172	457
	4	40	38	116	89	244	147	240	144	427	205	670	261
4.5	2	81	77	231	175	481	282	465	272	815	379	1243	466
	4	42	40	123	92	259	152	254	149	453	210	710	266
5	2	86	81	244	181	507	290	490	280	859	387	1310	473
	4	45	42	129	96	273	156	268	153	477	215	749	270

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-19 Bridge Drain Type B
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
4% Cross Slope (0.48"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	36	36	83	83	148	148	145	145	226	137	331	175
	4	23	23	54	54	80	56	77	54	136	82	207	109
1	2	36	36	98	80	198	148	187	128	320	187	469	239
	4	23	23	55	54	113	78	110	75	192	112	293	149
1.5	2	44	42	120	97	243	164	229	155	392	227	574	282
	4	24	23	67	54	139	94	134	91	235	136	359	176
2	2	50	48	139	111	281	189	264	178	453	252	663	308
	4	27	26	77	62	160	108	155	104	272	151	414	192
2.5	2	56	54	155	124	314	206	295	194	506	270	741	328
	4	30	29	86	69	179	118	173	114	304	162	463	205
3	2	62	59	170	136	344	219	324	206	554	285	812	344
	4	33	32	95	75	196	125	190	121	333	171	507	215
3.5	2	67	63	184	144	371	230	349	216	599	297	877	356
	4	36	34	102	80	212	131	205	127	359	178	548	222
4	2	71	68	196	151	397	239	374	225	640	307	938	365
	4	38	36	109	84	227	136	219	132	384	184	586	228
4.5	2	76	72	208	157	421	247	396	232	679	315	994	373
	4	41	39	116	87	241	141	232	136	407	189	622	233
5	2	80	76	220	163	444	254	418	239	716	322	1048	378
	4	43	41	122	90	254	145	245	140	430	193	655	236

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-20 Bridge Drain Type B
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
6% Cross Slope (0.72"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	65	65	152	125	316	220	305	213	535	321	816	428
	4	42	42	97	97	176	176	173	173	297	178	466	244
1	2	76	72	215	174	446	304	431	294	756	436	1153	559
	4	42	42	114	97	240	176	236	173	420	242	659	320
1.5	2	93	88	263	211	547	360	528	347	926	496	1413	627
	4	48	46	139	112	294	194	289	190	514	275	807	358
2	2	107	102	304	240	631	394	610	381	1069	537	1631	671
	4	56	53	161	127	340	212	334	208	594	298	932	383
2.5	2	119	113	340	259	706	420	682	406	1196	566	1824	700
	4	62	59	180	137	380	226	373	222	664	315	1042	400
3	2	131	124	372	275	773	441	747	425	1310	587	1998	717
	4	68	65	197	145	416	237	409	233	728	326	1142	410
3.5	2	141	132	402	287	835	457	806	441	1415	602	2158	726
	4	73	69	213	152	450	246	441	241	786	334	1233	415
4	2	151	138	430	298	893	469	862	453	1512	611	2307	728
	4	79	72	227	158	481	252	472	248	840	339	1318	416
4.5	2	160	144	456	307	947	478	914	462	1604	616	2447	725
	4	83	75	241	163	510	258	500	253	891	342	1398	414
5	2	169	149	480	315	998	485	964	469	1691	618	2579	717
	4	88	77	254	167	538	261	527	256	939	343	1474	410

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-21 Bridge Drain Type B
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
6% Cross Slope (0.72"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	65	65	149	149	276	193	260	182	446	268	652	342
	4	42	42	76	62	158	110	152	106	267	161	408	214
1	2	70	67	193	156	391	266	368	251	630	364	923	447
	4	42	42	107	87	223	152	216	147	378	218	577	280
1.5	2	86	82	237	190	478	315	450	296	772	413	1130	502
	4	46	44	132	106	273	180	264	174	463	248	706	314
2	2	99	94	273	216	552	345	520	325	891	447	1305	537
	4	53	51	152	120	316	197	305	190	535	268	816	336
2.5	2	111	105	306	233	618	368	581	346	996	472	1459	560
	4	60	57	170	130	353	210	341	203	598	283	912	350
3	2	122	115	335	247	677	386	637	363	1091	489	1598	573
	4	65	62	186	137	387	220	373	213	655	294	999	358
3.5	2	131	123	362	259	731	399	688	376	1179	501	1726	581
	4	71	66	201	144	418	228	403	220	707	301	1079	363
4	2	140	128	387	268	781	410	735	386	1260	509	1845	582
	4	76	69	215	149	446	234	431	226	756	306	1153	364
4.5	2	149	134	410	276	829	419	780	394	1337	514	1957	580
	4	80	72	228	154	474	239	457	231	802	308	1223	362
5	2	157	138	432	283	874	425	822	400	1409	515	2063	573
	4	84	74	240	157	499	243	482	234	845	309	1290	358

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-22 Bridge Drain Type C
Maximum Bridge Drain Spacing
Normal Crown Deck
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	1	20	20	45	45	88	53	83	50	142	72	208	92
	2	13	13	31	31	55	55	53	53	85	43	130	57
1	1	22	20	62	45	125	71	117	67	201	97	295	122
	2	13	13	34	31	71	55	69	53	121	58	184	77
1.5	1	27	24	76	52	153	86	144	81	246	116	361	147
	2	15	13	42	31	87	55	84	53	148	70	226	92
2	1	32	27	87	60	176	98	166	92	285	133	417	168
	2	17	15	48	33	101	56	97	54	171	80	260	105
2.5	1	35	30	98	66	197	109	186	103	318	148	466	186
	2	19	16	54	37	113	62	109	60	191	89	291	116
3	1	39	33	107	72	216	119	203	112	348	161	510	203
	2	21	18	59	40	123	68	119	66	209	97	319	127
3.5	1	42	36	115	78	233	128	220	120	376	173	551	215
	2	23	19	64	43	133	73	129	71	226	104	345	134
4	1	45	38	123	83	250	137	235	128	402	185	589	225
	2	24	21	69	46	143	78	138	75	241	111	368	140
4.5	1	48	40	131	88	265	144	249	136	427	192	625	233
	2	26	22	73	49	151	83	146	80	256	115	391	146
5	1	50	43	138	93	279	152	263	143	450	199	659	241
	2	27	23	77	51	159	87	154	84	270	120	412	151

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-23 Bridge Drain Type C
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	13	13	31	31	55	55	53	53	85	43	130	57
	4	9	9	20	20	36	36	36	36	47	24	74	33
1	2	13	13	34	31	71	55	69	53	121	58	184	77
	4	9	9	20	20	38	36	38	36	67	32	105	44
1.5	2	15	13	42	31	87	55	84	53	148	70	226	92
	4	9	9	22	20	47	36	46	36	82	39	129	52
2	2	17	15	48	33	101	56	97	53	171	80	260	105
	4	9	9	26	20	54	36	53	36	95	44	149	60
2.5	2	19	16	54	37	113	62	109	60	191	89	291	116
	4	10	8	29	20	61	36	60	36	106	49	166	66
3	2	21	18	59	40	123	68	119	66	209	97	319	127
	4	11	9	31	21	66	37	65	36	116	54	182	72
3.5	2	23	19	64	43	133	73	129	71	226	104	345	134
	4	12	10	34	23	72	39	70	39	125	58	197	77
4	2	24	21	69	46	143	78	138	75	241	111	368	140
	4	13	11	36	24	77	42	75	41	134	62	210	80
4.5	2	26	22	73	49	151	83	146	80	256	115	391	146
	4	13	11	39	26	81	44	80	44	142	64	223	83
5	2	27	23	77	51	159	87	154	84	270	120	412	151
	4	14	12	41	27	86	47	84	46	150	66	235	86

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-24 Bridge Drain Type C
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
2% Cross Slope (0.24"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	13	13	31	31	55	55	53	53	71	36	104	46
	4	9	9	20	20	36	36	24	15	43	22	65	29
1	2	13	13	31	31	62	55	59	53	101	49	147	61
	4	9	9	20	20	36	36	34	20	60	29	92	38
1.5	2	14	13	38	31	76	55	72	53	123	58	180	73
	4	9	9	21	20	44	36	42	24	74	35	113	46
2	2	16	14	44	30	88	55	83	53	142	66	208	84
	4	9	9	24	20	50	36	49	27	85	40	130	52
2.5	2	18	15	49	33	99	55	93	53	159	74	233	93
	4	10	9	27	20	56	36	54	30	95	44	146	58
3	2	19	17	53	36	108	59	102	56	174	80	255	101
	4	10	9	30	20	62	36	60	33	105	48	159	63
3.5	2	21	18	58	39	117	64	110	60	188	87	276	107
	4	11	10	32	22	67	37	64	35	113	52	172	67
4	2	22	19	62	42	125	68	117	64	201	92	295	112
	4	12	10	34	23	71	39	69	38	121	55	184	70
4.5	2	24	20	65	44	132	72	125	68	213	96	313	117
	4	13	11	36	24	76	41	73	40	128	58	195	73
5	2	25	21	69	46	139	76	131	72	225	100	329	120
	4	13	11	38	26	80	43	77	42	135	60	206	75

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-25 Bridge Drain Type C
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
4% Cross Slope (0.48"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	36	36	83	83	160	95	155	92	272	136	414	179
	4	23	23	54	54	98	98	97	97	151	76	237	103
1	2	38	36	109	83	227	129	219	124	384	184	586	241
	4	23	23	58	54	122	98	120	97	213	102	335	138
1.5	2	47	40	134	91	278	155	268	150	470	221	718	283
	4	24	23	71	54	150	98	147	97	261	123	410	161
2	2	54	46	154	105	321	178	310	171	543	245	829	308
	4	28	24	82	55	173	98	169	97	302	136	474	176
2.5	2	61	52	173	117	359	193	346	187	607	262	927	328
	4	32	27	91	62	193	104	189	102	337	145	529	187
3	2	66	57	189	127	393	205	379	198	665	276	1015	343
	4	35	29	100	67	212	110	208	108	370	153	580	196
3.5	2	72	61	204	135	424	215	410	208	719	287	1096	355
	4	37	32	108	72	229	116	224	114	399	160	626	203
4	2	77	65	218	142	454	223	438	216	768	297	1172	364
	4	40	34	116	75	244	120	240	118	427	165	670	208
4.5	2	81	69	231	147	481	231	465	223	815	305	1243	371
	4	42	36	123	78	259	124	254	122	453	169	710	212
5	2	86	73	244	152	507	237	490	229	859	311	1310	376
	4	45	38	129	80	273	128	268	125	477	173	749	215

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-26 Bridge Drain Type C
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
4% Cross Slope (0.48"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	36	36	83	83	148	148	145	145	226	113	331	144
	4	23	23	54	54	80	47	77	46	136	68	207	90
1	2	36	36	98	83	198	148	187	145	320	153	469	193
	4	23	23	55	54	113	64	110	62	192	92	293	121
1.5	2	44	38	120	82	243	148	229	145	392	184	574	226
	4	24	23	67	54	139	78	134	75	235	110	359	141
2	2	50	43	139	94	281	155	264	146	453	204	663	247
	4	27	23	77	54	160	89	155	86	272	122	414	154
2.5	2	56	48	155	105	314	169	295	159	506	218	741	262
	4	30	26	86	58	179	97	173	93	304	131	463	164
3	2	62	53	170	115	344	179	324	169	554	230	812	274
	4	33	28	95	64	196	103	190	99	333	138	507	171
3.5	2	67	57	184	122	371	188	349	177	599	240	877	284
	4	36	31	102	68	212	107	205	104	359	144	548	177
4	2	71	61	196	127	397	196	374	184	640	247	938	291
	4	38	33	109	71	227	112	219	108	384	148	586	182
4.5	2	76	64	208	132	421	202	396	190	679	254	994	297
	4	41	35	116	74	241	115	232	111	407	152	622	186
5	2	80	68	220	137	444	207	418	195	716	259	1048	301
	4	43	36	122	76	254	119	245	114	430	156	655	188

F = First Drain Spacing (ft) from crest
N = Subsequent Drain Spacing (ft)

Table 4-27 Bridge Drain Type C
Maximum Bridge Drain Spacing
Superelevated Deck Excluding Elevated Shoulder
6% Cross Slope (0.72"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	65	65	152	108	316	186	305	179	535	266	816	350
	4	42	42	97	97	176	176	173	173	297	148	466	200
1	2	76	65	215	148	446	252	431	244	756	356	1153	451
	4	42	42	114	97	240	176	236	173	420	198	659	258
1.5	2	93	79	263	179	547	297	528	286	926	402	1413	504
	4	48	42	139	97	294	176	289	173	514	224	807	288
2	2	107	91	304	203	631	324	610	313	1069	435	1631	538
	4	56	47	161	108	340	176	334	173	594	241	932	307
2.5	2	119	102	340	219	706	345	682	333	1196	457	1824	559
	4	62	53	180	116	380	186	373	182	664	254	1042	320
3	2	131	111	372	232	773	361	747	349	1310	474	1998	573
	4	68	58	197	123	416	195	409	191	728	263	1142	327
3.5	2	141	118	402	242	835	374	806	361	1415	485	2158	579
	4	73	62	213	128	450	201	441	198	786	270	1233	331
4	2	151	124	430	251	893	384	862	371	1512	492	2307	581
	4	79	64	227	133	481	207	472	203	840	274	1318	332
4.5	2	160	129	456	259	947	392	914	378	1604	496	2447	578
	4	83	67	241	137	510	211	500	207	891	276	1398	330
5	2	169	133	480	265	998	397	964	383	1691	498	2579	571
	4	88	69	254	140	538	214	527	210	939	277	1474	327

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

Table 4-28 Bridge Drain Type C
Maximum Bridge Drain Spacing
Superelevated Deck Including Elevated Shoulder
6% Cross Slope (0.72"/ft)

Grade (%)	Lanes Drained	Shoulder Width/Flooded Width (ft)											
		2/2		3/3		4/4		5/4		6/5		8/6	
		F	N	F	N	F	N	F	N	F	N	F	N
0.5	2	65	65	149	149	276	162	260	153	446	222	652	280
	4	42	42	76	54	158	93	152	90	267	133	408	175
1	2	70	65	193	149	391	221	368	208	630	297	923	361
	4	42	42	107	74	223	126	216	122	378	178	577	226
1.5	2	86	74	237	162	478	260	450	244	772	335	1130	403
	4	46	42	132	90	273	148	264	143	463	201	706	252
2	2	99	85	273	183	552	284	520	267	891	362	1305	430
	4	53	46	152	102	316	162	305	157	535	217	816	269
2.5	2	111	95	306	197	618	302	581	284	996	381	1459	448
	4	60	51	170	109	353	173	341	167	598	229	912	280
3	2	122	103	335	208	677	316	637	298	1091	395	1598	458
	4	65	56	186	116	387	181	373	174	655	237	999	286
3.5	2	131	110	362	218	731	327	688	308	1179	404	1726	463
	4	71	59	201	121	418	187	403	181	707	243	1079	290
4	2	140	115	387	226	781	336	735	316	1260	410	1845	465
	4	76	62	215	126	446	192	431	185	756	246	1153	290
4.5	2	149	120	410	233	829	343	780	322	1337	414	1957	462
	4	80	64	228	129	474	196	457	189	802	248	1223	289
5	2	157	124	432	239	874	348	822	327	1409	415	2063	457
	4	84	67	240	133	499	199	482	192	845	249	1290	286

F = First Drain Spacing (ft) from crest
 N = Subsequent Drain Spacing (ft)

4.10 Utilities

MaineDOT allows utility attachments to bridge structures if the proposed addition is a practical arrangement and is considered to be in the public interest. The design and construction of any additional structural supports and other appurtenances is the responsibility of the respective utilities. The location of the utility attachment should be selected to avoid conflict with existing utilities and with future utilities for which provisions have been made.

The Designer should consider the need for adequate access for maintenance and inspection. A minimum clear distance of 12 inches from any point on the main load carrying members and substructure units should be maintained. In addition, a minimum 2 foot clearance should be provided on at least one side of any utility attachment located between beams to allow access for future maintenance activities.

For buried structures, utilities must be located and designed to allow easy replacement of the buried structure. The preferred location for utilities is at the edge of the right-of-way or at least 15 feet from the end of the structure. If the utility must be buried in the roadway, a 12 inch clearance from the structure to the utility is required.

Utilities should not be located within the reinforcement limits of MSE walls or anchored wall systems.

The proposed installation must not decrease the underclearance of the structure. When locating utility attachments, consideration must also be given to aesthetics and the possibility of collision.

Where utility attachment is anticipated on prestressed superstructures, threaded inserts should be cast in the beams. Drilling for inserts will not be permitted. Utilities may be placed on the fascia of adjacent box and voided slab structures, under sidewalk utility bays, or between beams. Under no circumstances will utilities be allowed to pass through the internal voids of prestressed beams.

For girder bridges, utilities should be carried between the beams. The utilities should be supported by the steel or prestressed concrete framing system and not by the concrete deck. This requirement is to facilitate future deck replacement.

In general, conduits for electricity, telephone, or cable television should be located in the same manner as other utilities. In certain cases, these conduits may be embedded in the concrete sidewalk. Conduits may not be placed in the load-carrying portion of the structural slab. Conduits must be spaced to allow a 2 inch clearance from formwork, reinforcing steel, and other conduits.

For additional guidance, the Designer should consult Bridge Maintenance, the Utility Coordinator, and the Maine Utility Accommodation Policy located at the following link: <http://www.maine.gov/mdot/utilities/uap.php>.

4.11 Bearings

4.11.1 General

Bridge bearings should accommodate the movements of the superstructure and transfer the superstructure loads to the substructure. The type of bearing is dependent upon the magnitude/type of movement and the size of the applied loads.

Generally, the movements of the superstructure and the loads transferred to the substructure can be accommodated by elastomeric bearings. The Department's policy for bearings on new superstructures is to use elastomeric bearings wherever possible.

In some cases, structures with large bearing loads and/or multi-directional movements may require the use of pot or disc-type bearings, also known as floating bearings. Plans should direct which of these types to use, or whether interchanging types is intended. The use of spherical bearings may be necessary in more unique situations.

All elements of the bridge seat and bearing areas should be designed with maintenance in mind. In general, the vicinity of the bearing should be designed such that debris will not collect easily and provisions are made for bearing cleaning, repair, and replacement. Bearing repairs can be facilitated by using a bearing-to-masonry plate connection that can be readily removed, such as a weld or separate pin screw. The bearing area should be designed to allow inspection with reasonable effort.

Hold downs should be used when there is a concern for uplift revealed from the seismic analysis, or where stream or ice forces may act on the superstructure. Seismic sensitivity alone is not a requirement for hold downs.

The Structural Designer should become familiar with the Standard Specifications Section 523 - Bearings, as well as applicable manufacturer's product information, before specifying bearings for a particular project.

In addition to AASHTO LRFD Bridge Design Specifications, the NSBA references listed at the end of this chapter should be used as applicable.

4.11.2 Elastomeric Pads

The design of plain elastomeric bearings is in accordance with AASHTO LRFD Section 14. The use of cotton duck and fiberglass reinforced pads is not allowed.

Plain elastomeric bearings should be used for all precast concrete box beam and voided slab bridges. The standard dimensions of the plain elastomeric pad are given in Standard Detail 535 (01). For skewed bridges over 25° , consideration should be given to using circular bearing pads to reduce the contact pressure at the acute corner of the precast units.

4.11.3 Steel-Reinforced Elastomeric Bearings

The following is taken from the AISI/NSBA Guide Specification (draft 2003) and adapted to Maine's requirements.

The design of steel-reinforced elastomeric bearings should be in accordance with AASHTO LRFD Section 14. Design Method A is the preferred method for the design of elastomeric bearings, since it is less complicated and has fewer testing requirements. Bearings designed using Method A have an excellent performance history. The use of non-laminated elastomeric bearings is acceptable only if the design computations support their use. Design Method B will be used only for specialized cases.

Elastomeric bearings should be designed with the materials properties of either a 50 or 60 durometer neoprene or natural rubber material. The range in shear modulus for design is 100 to 130 psi. The steel reinforcing shims should meet ASTM A36. All other steel components except anchor bolts should meet the requirements of AASHTO M270, Grade 50W (refer to Appendix D Standard Notes Elastomeric Bearings).

The plans should also state the unfactored dead load, live load, and total reactions in the longitudinal and transverse directions, along with the total required movements for each elastomeric bearing design.

4.11.3.1 Design Rotation

In general, elastomeric bearings should be designed for unfactored live load rotation and additional rotations due to uncertainties and construction tolerances. Dead load rotation should only be added to the design rotation when a beveled sole plate is not used. Refer to Section 4.11.3.4 Sole Plate Details for further guidance.

The bearing should also be designed for an additional rotation of 0.005 radians to account for construction tolerances and uncertainties.

4.11.3.2 Design Movement

The design movement should allow for welding of the sole plate to the girder at temperatures in the range of 60°F to 90°F. Refer to Section 3.3 Thermal Effects for the applicable temperature range.

4.11.3.3 Masonry Plates

Masonry plates should be used under fixed and expansion bearings supporting both steel and prestressed concrete girders. Masonry plates for expansion bearings should have a minimum thickness of 1-1/4" with a 1/4" recess for the bearing. For fixed bearings the minimum thickness should be 1 inch. The minimum dimensions should not be less than the sole plate dimensions as specified in Section 4.11.3.4. Masonry plates should be vulcanized to the bearing during the primary molding process and should be hot-dip galvanized or metallized (refer to Appendix D Standard Notes Elastomeric Bearings).

4.11.3.4 Sole Plate Details

Sole plates should be used for fixed bearings supporting both steel and prestressed concrete girders and expansion bearings supporting steel girders. For bearings supporting prestressed girders, sole plates should be used only if a beveled surface is required to account for camber or the profile grade.

The sole plate should extend transversely beyond the edge of the bottom flange of the girder a minimum of 1 inch on each side. The minimum thickness of the sole plate should be 1-1/2" after beveling if the weld is directly over the elastomer. Beveled plates as thin as 3/4" minimum may be used if there is a lateral separation of at least 1-1/2" between the weld and the elastomer.

Sole plates should be beveled to account for all dead load rotations and grade differences at the bearing. A beveled sole plate should be used when the slope of the girder at the centerline of bearing exceeds 1.0%. The bevel of the sole plate should match the slope of the girder as near as possible and be depicted on the contract drawings. Sole plates should not be beveled if the total change in thickness of the sole plate is less than 1/4". In this case, the dead load rotations and rotations due to grade differences should be included in the design rotation.

4.11.3.5 Bearing to Girder Connection

The bearing may be connected to the girder by field welding or field bolting. For connections designed with welds, the welds should be in the horizontal

position. The welds for the sole plate connection should be located only along the longitudinal girder axis. Transverse joints should be sealed with an acceptable caulking material. The bearing should be detailed with at least 1-1/2" of steel between the elastomer and any field welds.

The elastomer is at risk for damage during the welding process. Refer to Appendix D Standard Notes Elastomeric Bearings for an appropriate note.

A 1/2" steel plate with shear studs should be cast into the bottom flange of New England Bulb Tees (NEBT) and AASHTO I-girders to allow for welding to the sole plate.

4.11.3.6 Lateral and Uplift Restraint

Anchor rods should be used for fixed bearings to resist the lateral and uplift forces, if applicable, acting on the bearing. For expansion bearings, anchor rods are required to act as hold downs if uplift forces are present. Lateral forces should be resisted by keeper angles when hold downs are not required. For bridges that are very wide, or with high skews, care should be taken with the orientation of the slotted holes in the sole plate or the keeper angles. Skewed bridges will tend to expand along an axis that runs from acute corner to acute corner. Bridges that are wider than they are long will expand more in the transverse direction than in the longitudinal direction.

4.11.3.7 Anchor Rods

The design of anchor rods for lateral load should consider the bending capacity of the rod, edge distance to the concrete foundation, strength of the concrete, and group action of the rods. Material for anchor rods should be ASTM F1554, and swedged on the embedded portion of the rod. The design yield strength of this material can either be 55 ksi or 105 ksi, which should be noted on the plans.

4.11.3.8 Elastomeric Bearings with Sliding Surfaces

Sliding surface bearings should be used only for situations where the combined effects of large movement and low load do not permit the economical use of conventional elastomeric bearings. Anchor rods should be used only on this bearing type when there is a concern for uplift, or where stream or ice forces may act on the superstructure. Anchor rods if used, should be investigated for the combined effects of shear and bending. A shear plate may be incorporated into the design to reduce the bending effects in the anchor rods.

4.11.3.9 Marking

To ensure the proper orientation of the bearing during placement, use the appropriate note from Appendix D Standard Notes Elastomeric Bearings.

4.11.4 High Load Multi-Rotational Bearings

The following is taken from the AISI/NSBA Guide Specification (draft 2003) and adapted to MaineDOT's requirements.

There are three common High Load Multi-Rotational (HLMR) bearing types that function in essentially the same manner. They include pot bearings, disc bearings, and spherical bearings. The AASHTO design specifications give detailed guidance for the design and manufacture of these bearings. All three types of HLMR bearings should be allowed on most projects; however, a project specific special provision will be needed for spherical bearings stating the design and fabrication guidelines.

Contract plans for bridges with HLMR bearings should not include specific details for the bearings, since the manufacturer designs the bearing. The Structural Designer should specify the type of bearing on the plans (pot, disc, spherical, or choice of more than one). Only schematic bearing details combined with specified loads, longitudinal and transverse movements, and rotations, as well as fixed/expansion types should be shown. A bearings setting table should also be included on the plans. Designers are expected to review these designs during the shop drawing review process.

All steel excluding anchor bolts should be AASTHO M 270, Grade 50 W.

4.11.4.1 Design Recommendations

The design of HLMR bearings is the responsibility of the bearing manufacturer. The design of accessory pieces of the bearing, such as the sole plate, masonry plate, and the anchor rods, are the responsibility of the manufacturer in accordance with Standard Specifications Section 523 - Bearings. However, the Structural Designer may design the accessories or a portion of the accessories if they so choose.

4.11.4.2 Uplift Forces

If the bearings do not require design for uplift forces, this should be noted on the plans. Refer to Appendix D Standard Notes HLMR Bearings.

4.11.4.3 Design Rotation

In general, HLMR bearings should be designed for factored live load rotation and additional rotations for uncertainties and construction tolerances. Dead load rotation should be added to the design rotation only when a beveled sole plate is not used. The bearing should also be designed for an additional rotation of 0.01 radians to account for construction tolerances and uncertainties. The contract drawings should clearly state whether or not the additional 0.01 radians for construction tolerances and uncertainties is included in the design rotation stated on the contract drawings.

4.11.4.4 Design Movement

Refer to Section 3.3 Thermal Effects for the applicable temperature range.

4.11.4.5 Sole Plate

The preferred connection of the HLMR sole plate to I-girder is field welding. Connection to steel box girders should be bolted. The sole plate should extend transversely beyond the edge of the bottom flange of I-girders a minimum of 1 inch on each side. Welds for sole plate connections should be located only longitudinal to the girder axis. Transverse joints should be sealed with an approved caulking. The minimum thickness of the sole plate is 3/4".

4.11.4.6 Future Maintenance

HLMR bearings should be designed for future removal with a maximum vertical jacking height of 1/4" after the load is removed. The minimum distance between the bottom of the masonry plate to the top of the sole plate should be 4 inches. This requirement should be addressed in a special provision in the PS&E package.

4.11.4.7 Masonry Plate and Anchor Rods

The masonry plate should bear directly on a 1/8" thick preformed elastomeric pad that rests directly on the substructure. The location of anchor rods should allow for future bearing removal. For anchor rod design, refer to Section 4.11.3.7.

4.11.4.8 Marking

To ensure the proper orientation of the bearing during placement, use the appropriate note from Appendix D Standard Notes HLMR Bearings.

4.11.4.9 Pot Bearings and Disk-type Bearings

A. 4.1.3.1 PTFE Material

PTFE (Teflon) material used in vertical sliding surfaces (i.e., retaining bars) should be fastened with at least two methods as specified in AASHTO; however, MaineDOT requires one of these methods to be countersunk screws.

B. Height Considerations

An initial distance between the concrete bearing seat and the underside of the bottom beam flange ("H") should be established by the Structural Designer and placed on the plans. After the Contractor or Subcontractor designs the pot or disk-type bearing, an adjustment is established and conveyed to the Resident so that modifications may be made to the final bearing seat elevations. Generally, disk-type bearings have a lower profile than pot bearings.

4.11.5 *Steel Bearings*

The use of steel rocker bearings is not recommended. Steel fixed bearings as described in Section 3 of the AISI/NSBA Guide Specification (draft 2003) may be used if they are more economical than elastomeric fixed bearings.

References

AASHTO, 1990, *A Guide for Protective Screening of Overpass Structures*

AASHTO, 1984, *An Informational Guide for Roadway Lighting*

AASHTO, 1998 and Interims, *Load and Resistance Factor Design (LRFD) Bridge Design Specifications*, Washington, DC

AISI/NSBA Steel Bridge Collaboration Task Group 9, 2002 (draft), *Guide Specification for Steel Bridge Bearing Design and Detailing*, December 10

AISI/NSBA, 1996, *Steel Bridge Bearing Selection and Design Guide*, December

Federal Highway Administration, *Drainage of Highway Pavements*, FHWA Hydraulic Engineering Circular No. 12 (HEC 12)

Federal Highway Administration, *Design of Bridge Deck Drainage*, FHWA Hydraulic Engineering Circular No. 21 (HEC 21)

Chapter 5

SUBSTRUCTURES



Smith Bridge, Houlton



West Bridge, Fairfield-Benton

5	SUBSTRUCTURES	5-1
5.1	Terminology	5-1
5.2	General	5-3
5.2.1	Frost	5-3
5.2.2	Seal Cofferdams	5-6
5.2.3	Cofferdams	5-7
5.2.4	Concrete Joints	5-8
5.2.5	Seismic Considerations	5-9
5.3	Spread Footings	5-10
5.3.1	Service Limit States	5-10
5.3.2	Strength Limit States	5-11
5.3.3	Extreme Event Limit States	5-12
5.3.4	Footing Depth	5-12
5.3.4.1	Bearing Materials	5-12
5.3.4.2	Footings on Bedrock	5-13
5.3.4.3	Frost Protection	5-13
5.3.4.4	Scour Protection	5-13
5.3.5	Bearing Resistance	5-13
5.3.5.1	General	5-13
5.3.5.2	Bearing Stress Distribution	5-14
5.3.5.3	Bearing Resistance Factors	5-15
5.3.6	Settlement	5-16
5.3.6.1	Tolerable Settlement	5-17
5.3.6.2	Settlement Analyses	5-17
5.3.7	Overall Stability	5-18
5.3.8	Sliding	5-18
5.3.9	Eccentricity	5-20
5.3.10	Ground Water Condition	5-20
5.3.11	Drainage Considerations	5-20
5.4	Abutments	5-20
5.4.1	Conventional Abutments	5-20
5.4.1.1	General Design Requirements	5-20
5.4.1.2	Loads Combinations and Load Factors	5-21
5.4.1.3	General	5-23
5.4.1.4	Strength Limit State Evaluations	5-24
5.4.1.5	Service Limit State Evaluations	5-24
5.4.1.6	Extreme Limit State Evaluations	5-25
5.4.1.7	Load Considerations	5-25
5.4.1.8	Backfill	5-26
5.4.1.9	Drainage	5-26
5.4.1.10	Reinforcement and Structural Design	5-27
5.4.1.11	Abutments on Spread Footings	5-27
5.4.1.12	Abutments Supported on Pile Foundations	5-37
5.4.1.13	Bridge Seat Dimensions	5-38
5.4.2	Integral Abutments	5-38
5.4.2.1	Introduction	5-38

5.4.2.2	Loads	5-39
5.4.2.3	Historical ASD Design Practice and Bridge Lengths	5-39
5.4.2.4	Pile Design.....	5-40
5.4.2.5	Pile Length Requirement	5-43
5.4.2.6	Maximum Bridge Lengths	5-45
5.4.2.7	'Best Practices' for Moderate to Long Span IABs	5-46
5.4.2.8	Abutment Details.....	5-48
5.4.2.9	Alignment.....	5-51
5.4.2.10	Superstructure Design	5-51
5.4.2.11	Abutment and Wingwall Design	5-51
5.4.2.12	Approach Slabs.....	5-52
5.4.2.13	Drainage	5-53
5.4.2.14	Scour	5-53
5.4.2.15	Integral Abutment on Spread Footing Design	5-53
5.4.3	Semi-Integral Abutments	5-53
5.4.4	Approach Slabs	5-54
5.5	Piers	5-55
5.5.1	Mass Piers.....	5-55
5.5.1.1	Pier Selection Criteria	5-55
5.5.1.2	Load Combinations and Load Factors	5-55
5.5.1.3	General	5-58
5.5.1.4	Strength Limit State Evaluations	5-58
5.5.1.5	Service Limit State Evaluations	5-59
5.5.1.6	Extreme Event Limit State Evaluations	5-59
5.5.1.7	Structural Design	5-60
5.5.1.8	Structural Design of Columns	5-60
5.5.1.9	Geotechnical Design of Pier Foundations.....	5-61
5.5.1.10	Pier Protection	5-61
5.5.2	Pile Bent Piers	5-63
5.5.2.1	Pile Bent Use Criteria.....	5-63
5.5.2.2	Loads and Load Combinations	5-64
5.5.2.3	Pile Cap Design	5-67
5.5.2.4	Pile Type Selection Criteria.....	5-67
5.5.2.5	Pile Protection.....	5-68
5.5.2.6	Pipe Pile Coatings and Cathodic Protection	5-69
5.5.2.7	Additional Pile Bent Pier Design Criteria.....	5-71
5.6	Retaining Walls.....	5-75
5.6.1	General.....	5-75
5.6.1.1	Retaining Wall Type Selection	5-75
5.6.1.2	Service Life	5-76
5.6.1.3	Design Loads	5-76
5.6.1.4	Limit States	5-77
5.6.1.5	Strength Limit State	5-79
5.6.1.6	Service Limit State Checks	5-80
5.6.1.7	Design Considerations.....	5-81
5.6.1.8	Aesthetics	5-81

CHAPTER 5 - SUBSTRUCTURES

5.6.2	Gravity Retaining Walls.....	5-81
5.6.2.1	Design Section.....	5-81
5.6.2.2	Earth Loads	5-82
5.6.3	Gravity Cantilever-type Retaining Walls	5-82
5.6.3.1	Design Section Gravity Cantilever Retaining Walls.....	5-82
5.6.3.2	Earth Loads	5-83
5.6.4	Non-Gravity Cantilever and Anchored Retaining Walls	5-84
5.6.4.1	Soil Nail Walls.....	5-84
5.6.5	Prefabricated Proprietary Walls	5-84
5.6.5.1	Proprietary Retaining Walls	5-85
5.6.5.2	Prefabricated Concrete Modular Gravity Walls	5-85
5.6.5.3	Precast Concrete Block Gravity Walls	5-87
5.6.5.4	MSE Walls	5-87
5.6.5.5	PS&E for Project with Proprietary Walls	5-91
5.6.6	Geosynthetic Reinforced Soil Integrated Bridge Systems	5-92
5.6.7	Anchored Wall Systems.....	5-92
5.6.7.1	CON/SPAN® Wingwall.....	5-92
5.6.7.2	Metal Structural Plate Headwall/Wingwall.....	5-93
5.6.8	Gabions	5-93
5.7	Piles.....	5-94
5.7.1	General.....	5-94
5.7.2	H-Piles	5-94
5.7.2.1	Axial Resistance	5-94
5.7.2.2	Lateral Pile Resistance for the Service Limit State	5-96
5.7.3	Layout and Construction	5-99
5.7.4	Concrete Piles	5-99
5.7.5	Steel Pipe Piles.....	5-100
5.7.5.1	Design - General.....	5-100
5.7.5.2	Material and Design Section	5-100
5.7.6	Downdrag	5-101
5.7.7	Pile Installation Quality Control and Nominal Pile Resistance.....	5-102
5.8	Drilled Shafts	5-105
5.9	Embankment Issues	5-106
5.9.1	Embankment Settlement.....	5-106
5.9.2	Embankment Stability	5-106
5.9.3	Embankment Bearing Capacity	5-108
5.9.4	Embankment Seismic Considerations	5-108
	References	5-109
	Table 5-1 Depth of Frost Penetration.....	5-3
	Table 5-2 Bearing Resistance Factors.....	5-16
	Table 5-3 Resistance Factors for Sliding of Spread Footings at the Strength Limit State.....	5-19
	Table 5-4 Typical Load Groups and Load Factors (γ_i) for Abutments on Spread Footings.....	5-23

Table 5-5 Recommended Maximum Lengths for Fully Integral Abutment Bridges (feet)	5-46
Table 5-6 Typical Load Groups and Load Factors	5-79
Table 5-7 Factored Axial Structural Resistance of Selected H-Pile Sections ..	5-96
Table 5-8 Factored Lateral Resistance and Depth to Fixity for Strength Limit State Design for H-Pile Sections in Sand, $\phi=1.0$	5-97
Table 5-9 Factored Lateral Resistance and Depth to Fixity for Service Limit State Design for H-Pile Sections in Clay, $\phi=1.0$, Load Perpendicular to Flange	5-98
Table 5-10 Resistance Factors for Driven Piles	5-103
Figure 5-1 Maine Design Freezing Index Map	5-5
Figure 5-2 Fixed Pile Head, Full Integral Abutment Details-Steel Superstructures	5-49
Figure 5-3 Integral Abutment with Hinge and Full Integral Abutment Details – Precast Superstructures	5-50
Figure 5-4 Effective Pile Length for Piles in Sand	5-73
Figure 5-5 Effective Pile Length for Piles in Clay	5-74
Figure 5-6 Retaining Wall Design Section	5-83
Procedure 5-1 Bearing Stress on Soil	5-30
Procedure 5-2 Bearing Stress on Bedrock	5-32
Procedure 5-3 Eccentricity and Sliding Check for Conventional Abutment on Spread Footing	5-35
Example 5-1 Depth of Frost Penetration	5-6

5 SUBSTRUCTURES

5.1 Terminology

B	footing width
B'	effective footing width
C	point designating center of footing
D	height of soil in front of structure, which is applicable to passive resistance
DL_v, LL_v	vertical structural/superstructure loads applied to abutment wall
D_f	depth to fixity
e	eccentricity of the resultant of all vertical forces at the bottom of the footing, measured from mid-width of footing
e_o	eccentricity calculated about the toe of the footing, to be used for overturning calculations
E_p	modulus of elasticity of pile
E_g	modulus of elasticity of end span beam/girder
F.G.	finished grade elevation
H	height of structure or failure plane
H_t	horizontal force required to translate pile
I_p	moment of inertia of pile
I_g	moment of inertia of end span beam/girder (composite I for composite beams)
K	effective length factor
K_a	active earth pressure coefficients for level or sloped backfill
K_{ho}	active earth pressure coefficient corresponding to a broken backslope
K_o	at-rest earth pressure coefficient
K_p	passive earth pressure coefficient.
L	heel length
L'	effective footing length
L_e	effective pile length from ground surface to the point of assumed fixity below ground, including scour effects.
L_s	length of end span
L_u	exposed pile length above ground
L_{us}	unsupported length
M	pile head moment
M_o	overturning moment
M_r	resisting moment
M_t	moment induced in the pile from the horizontal translation
O	point designating the toe of footing
$P_{h,q}$	horizontal traffic surcharge force behind abutment wall
P_h	horizontal soil active force behind abutment wall
P_L	allowable lateral load
P_p	horizontal passive force
P_t	pile reaction resulting from the earth pressure on the abutment

CHAPTER 5 - SUBSTRUCTURES

q_s	traffic live load surcharge pressure
Q	factored horizontal sliding force
Q_{applied}	applied load or stress
R	resultant force at base of footing
R_n	nominal resistance of footing, pile, shaft or micropile
R_R	factored resistance of a footing, pile, shaft or micropile
R_f	factored bearing or sliding resistance of a footing
R_g	beam/girder rotation (radians)
S_p	section modulus of the pile
t	footing thickness
w	water content (percent)
W	total beam/girder live load, end span
W_{c1}, W_{c2}	weight of abutment wall, footing
W_s	weight of soil above heel
W_{toe}	weight of soil above toe
X_{DL}	distance from the point of interest to the dead load reaction (centerline of bearing)
X_{LL}	distance from the point of interest to the live load reaction (centerline of bearing)
X_{WS}	distance from the point of interest to the centroid of W_s
X_{WC1}	distance from the point of interest to the centroid of W_{c1}
X_{WC2}	distance from the point of interest to the centroid of W_{c2}
X_{wtoe}	distance from the point of interest to the centroid of W_{toe}
y	the depth of seal from top of seal to bottom of seal
z	the depth of water from water surface to bottom of seal
α	batter angle from the horizontal plane
β	backfill slope
δ	friction angle between soil/bedrock and concrete
γ	soil weight
λ	column slenderness factor
η_i	factors to account for ductility, redundancy and operational importance
γ_i	load factor (general)
γ_p	permanent load factor
ϕ	soil internal angle of friction
σ_v	factored bearing stress at base of footing
τ	horizontal superstructure forces transmitted through bearing at wall top
Φ_c	resistance factor for axial compression
Φ_f	resistance factor for flexure
ϕ	resistance factor (general - geotechnical)
ϕ_{bc}	resistance factor for bearing resistance
ϕ_{dyn}	resistance factor for driven piles, dynamic analysis methods
ϕ_{stat}	resistance factor for piles, static analysis methods
ϕ_{ep}	resistance factor for passive soil resistance
ϕ_{τ}	resistance factor for sliding resistance between footing and soil/rock

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

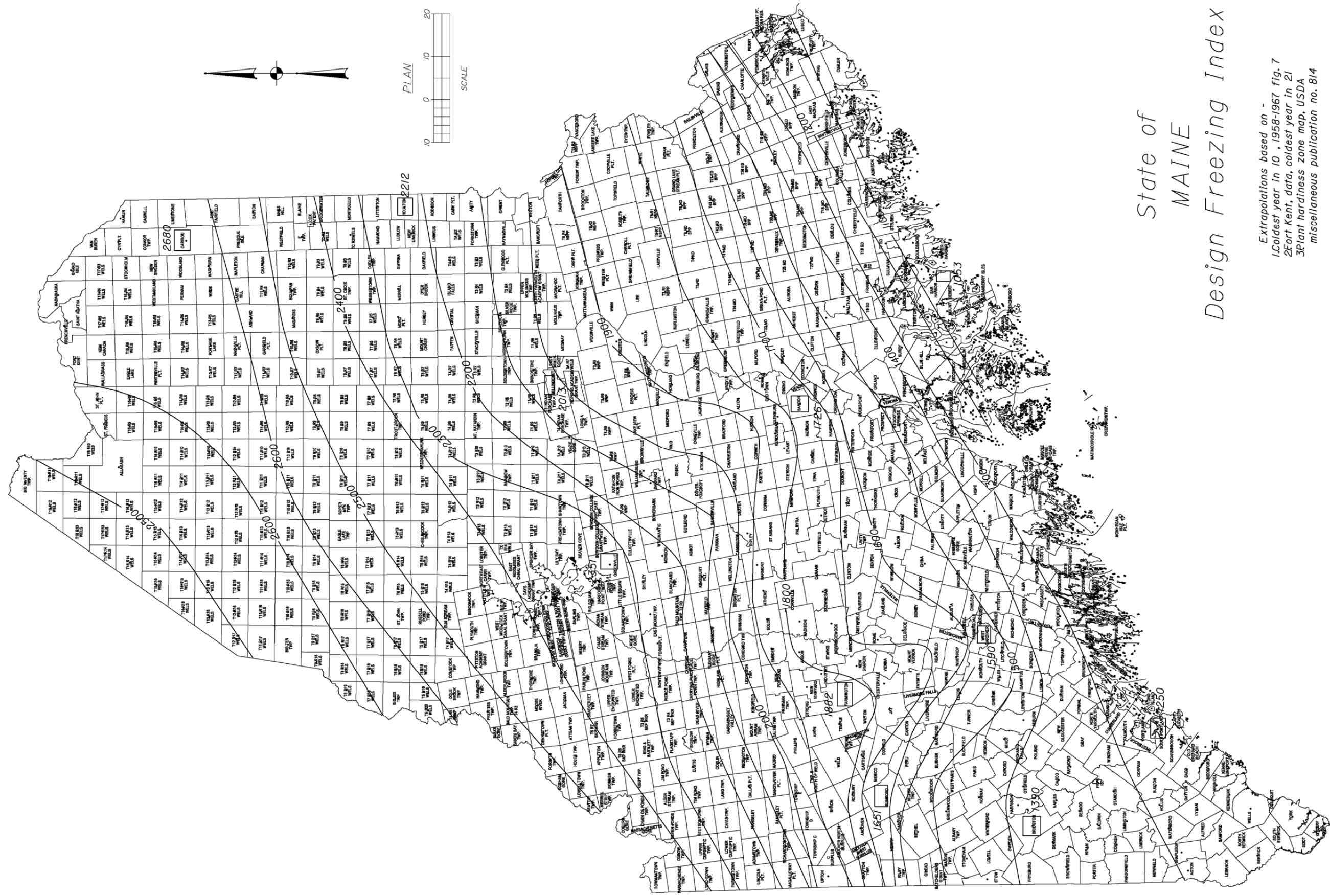
In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

CHAPTER 5 - SUBSTRUCTURES

- Notes:
1. w = water content
 2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.



State of
MAINE
Design Freezing Index

Extrapolations based on -
1. Coldest year in 10, 1958-1967 fig. 7
2. Fort Kent, data, coldest year in 21
3. Plant hardiness zone map, USDA
miscellaneous publication no. 814

Figure 5-1 Maine Design Freezing Index Map

Example 5-1 illustrates how to use Table 5-1 and Figure 5-1 to determine the depth of frost penetration:

Example 5-1 Depth of Frost Penetration

Given: Site location is Freeport, Maine
 Soil conditions: Silty fine to coarse Sand

- Step 1.** From Figure 5-1 Design Freezing Index = 1300 degree-days
- Step 2.** From laboratory results: soil water content = 28% and major constituent Sand
- Step 3.** From Table 5-1: Depth of frost penetration = 56 inches = 4.7 feet

Spread footings founded on bedrock require no minimum embedment depth. Pile supported footings will be embedded for frost protection. The minimum depth of embedment will be calculated using the techniques discussed in Example 5-1. Pile supported integral abutments will be embedded no less than 4.0 feet for frost protection.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

The final depth of footing embedment may be controlled by the calculated scour depth and be deeper than the depth required for frost protection. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.2.2 Seal Cofferdams

Seal cofferdams are used when a substructure unit must be constructed with its foundation more than 4 feet below the water table, to counteract the buoyant forces produced during pumping of the cofferdam. Once the cofferdam is constructed, the seal is placed under water and water is then pumped out of the cofferdam. This provides a dry platform for construction of the spread footing, or in the case of a pile foundation, the distribution slab. When a seal is needed, the top of footing or distribution slab is located approximately at streambed, and the depth of seal is calculated based upon the buoyancy of the concrete under the expected water surface during construction. The following formula can be used:

$$145 \cdot y = 62.4 \cdot z$$

where:

- 145 lb/ft³ = unit weight of concrete
- 62.4 lb/ft³ = unit weight of water
- y = the depth of seal from top of seal to bottom of seal
- z = the depth of water from water surface to bottom of seal

The depth of water in the above formula should be based on an appropriate flood event, but no less than Q10. The depth of water at tidal locations should be selected on a case-by-case basis, but no less than MHW. A note should be included on seal cofferdam sheets specifying the water elevation assumed in the design and specifying adjusting the seal depth should the water elevation at the time of construction be higher. To prevent seal buoyancy during a high water event after construction is complete, the Designer may specify vent holes at the design height of water, on a case-by-case basis.

Anchorage of the footing or distribution slab to the seal is required. For pile-supported foundations, this can be accomplished by extending the piles into the distribution slab. For seals founded on bedrock, dowels should be drilled and grouted into the seal after dewatering and prior to placement of the footing.

When sheet piling is used for a seal cofferdam, the minimum dimensions for the seal should be shown on the design drawings. These dimensions and details should be noted on the plans in conjunction with the appropriate notes in Appendix D Standard Notes Seal Cofferdams.

5.2.3 Cofferdams

Cofferdams are retaining structures with the retained material being water and soil. A separate cofferdam must be specified for the construction of each substructure unit (abutment or pier) that cannot be constructed completely in the dry. When water cannot be controlled so that footing concrete can be placed in the dry, a concrete seal must be placed below the elevation of the footing. Refer to Section 5.2.2 Seal Cofferdams.

Cofferdam design is the responsibility of the Contractor, and construction requirements are found in Standard Specification Section 511 – Cofferdams. Unless otherwise provided or approved, cofferdams are removed after the completion of the substructure, with care being taken not to disturb or otherwise damage the finished work.

Cofferdams should not be specified for substructure units that are constructed on dry land, such as on overpass structures. For large braced excavations a Special Provision should be included in the PS&E package to pay for braced excavations under the appropriate cofferdam item. Any temporary retaining structures that are required to support small structural excavations should be considered incidental to the appropriate structural excavation or substructure pay items.

Cofferdam requirements for culverts and other buried structures are found in Section 8.1.2 Construction Practices.

5.2.4 Concrete Joints

Concrete joints in a vertical plane are used in concrete construction to accommodate changes in the volume of concrete caused by such factors as drying shrinkage, creep, and the application of load. When concrete is restrained by internal or external forces, the stresses caused by concrete movement would be relieved by the formation of significant cracks, if joints were not provided. Construction joints are used to facilitate the sequence of construction, and are typically located in a horizontal plane for abutments, piers, and walls.

There are three types of joints commonly used in concrete construction. A concrete key is generally used with each joint for shear transfer, as shown in Standard Detail 502 (01). The Structural Designer should specify the proper concrete joint, depending upon its intended use.

- Contraction joints are normally used every 30 feet along a wall to control the location of cracks. Without these joints, the concrete would form cracks at unpredictable intervals. Reinforcing steel is normally not carried through the joint, except in rigid frame structures, where moment must be transferred from wall to slab.
- Expansion joints are used to prevent compression forces from abutting concrete from crushing or displacing the adjacent structure. It is good practice to locate expansion joints where expansion forces change direction, such as at wingwall turns. In retaining walls and abutment/wingwall systems, expansion joints should be spaced no more than 90 feet apart. Reinforcing steel is not carried through the joint.
- Construction joints are used between concrete placements when the sequence of construction requires more than one placement. The surface between placements becomes a construction joint. These joints may be designed to coincide with contraction or expansion joints. If not functioning as a contraction or expansion joint, reinforcing steel is normally carried through the joint.
- A horizontal construction joint in the abutment backwall should be shown on the plans to facilitate installation of the superstructure expansion device. This should normally be located at a minimum vertical distance of 1'-3" from the roadway surface, except for modular expansion devices, which must conform to the manufacturer's recommendations (refer to Section 4.8.5 Modular Joints). Bent #5 bars at 1'-6" maximum spacing should be used in the top of the backwall. Welding to reinforcing steel is allowed in this area so that the Contractor can utilize the reinforcing steel to support the expansion device.

5.2.5 *Seismic Considerations*

Seismic analysis of bridges and foundations shall be performed in accordance with the LRFD Specifications or the AASHTO Guide Specifications for LRFD Seismic Design (herein referred to as the Guide Specification).

Seismic analysis is not required for the following:

- Single-span bridges, regardless of Seismic Zone or Seismic Design Category (SDC)
- Any bridge in Seismic Zone 1 or SDC A, with the exceptions described below.

For all bridges, including those for which seismic analysis is not required, superstructure connections and bridge seat dimensions should be satisfied per LRFD 3.10.9 and 4.7.4.4, respectively.

For critical or essential bridges, including those in Seismic Zone 1 or SDC A, the Department may specify a higher Seismic Zone or SDC than that specified by the LRFD Specifications and the Guide Specification or specify appropriate seismic provisions. Critical and essential bridges are not specifically classified in this Bridge Design Guide, but will be designated as such by the Department at its discretion.

In general, bridges that may be classified by the Department as critical or essential are as follows:

- Bridges that are required to be open to all traffic once inspected after the design earthquake and usable by emergency vehicles and for security, defense, economic or secondary life safety purposes immediately after the design earthquake.
- Bridges that should be open to emergency vehicles and/or for security, defense or economic purposes after the design earthquake and open to all traffic within days after that event.
- Bridges that are formally designated as critical for a defined local emergency plan.

For non-conventional bridges, including cable-stayed and suspension bridges, truss bridges, arch type bridges and movable bridges the Department will specify and approve appropriate seismic design provisions.

It is estimated that most bridge sites in Maine will be classified as Seismic Zone 1 or SDC A. The exception are bridges in the extreme northwest portion where the subsurface conditions might be classified as Site Class B, C or D, and bridge sites everywhere where the subsurface conditions are Site Class

E, except those in downeast coastal Maine. It is estimated these bridge sites will be classified as Seismic Zone 2 or SDC B.

For bridges requiring seismic analysis, the effect of earthquake loading on the foundations shall be investigated using the extreme event limit state in LRFD Table 3.4.1-1 with resistance factors, ϕ , of 1.0 and an appropriate seismic analysis method as described in LRFD 4.7.4.3 and LRFD 3.10.9.2 through 3.10.9.4. The foundation design should consider the effect of wall inertia and amplification of active earth pressure by earthquake determined by the Mononobe-Okabe method. The Mononobe-Okabe method for determining equivalent static fluid pressure for seismic loads on walls is presented in LRFD 11.6.5 and Appendix A11. LRFD Appendix A10 gives additional guidance regarding seismic analysis and design of foundations.

For foundations on soil and rock, the location of the resultant of the reaction forces due to earthquake loading should be within the middle two-thirds (2/3) of the footing base for $\gamma_{EQ} = 0.0$ and within the middle eight-tenths (8/10) of the footing base for $\gamma_{EQ} = 1.0$. For in between values of γ_{EQ} , the restriction for the location of the resultant is obtained by linear interpolation of the preceding values of γ_{EQ} .

For overall stability of a retaining wall when earthquake loading is included, a resistance factor, ϕ , of 0.90 should be used. For bearing resistance, a resistance factor, ϕ , of 0.80 should be used for gravity and semigravity walls and 0.90 for MSE walls.

Where the backfill or foundation soils are saturated, consideration should be given to address the possibility of soil liquefaction and lateral spreading. Liquefaction design guidance is provided in LRFD 10.5.4.2, 11.5.4.2 and Appendix A10.

5.3 Spread Footings

Spread footings should be designed and proportioned for the strength, service, and extreme event limit states such that the factored resistance is not less than the effects of the factored loads specified in LRFD Article 3.

Selection of foundation type is based on an assessment of the magnitude and direction of loading, depth to suitable bearing materials, flood history, potential for liquefaction, undermining, scour or wave action, frost depth, and ease and cost of construction.

5.3.1 Service Limit States

Spread footings at the service limit state shall be investigated for:

- Settlement

- Horizontal movement
- Rotation
- Overall stability of slope with the footing
- Scour at the design flood, specified in LFRD 2.6.4.4.2 and 3.7.5

Settlement shall be investigated for the Service I Load Combination and rotations and horizontal movements shall be investigated at all applicable service limit states.

The tolerable level of ultimate settlement, differential settlement, rotation and horizontal movement shall be controlled by superstructure tolerance, rideability, span length, road classification, long-term maintainability and economy.

Bearing resistance estimated using presumptive allowable bearing resistances shall only be applied to address service limit state load combinations or for preliminary sizing of footings.

Service limit state analyses shall use unfactored loads. Resistance factors for the service limit state shall be taken as 1.0. The exception is the investigation of the overall slope stability of a retaining wall or an earth slope supporting a retaining wall footing or an abutment footing. In those instances, the earth slopes should be investigated at the Service I Load Combination, with a resistance factor, ϕ , of 0.65.

5.3.2 *Strength Limit States*

The design of spread footings at the strength limit states shall consider:

- Factored bearing resistance
- Eccentricity or loss of contact
- Sliding
- Loss of lateral and vertical support due to scour at the design flood event; the design flood is defined as the more severe of the 100-year even or an overtopping flood of lesser recurrence interval.
- Factored structural resistance

Resistance factors for the bearing resistance of spread footings at the strength limit state are provided in Section 5.3.5.3. Resistance factors for sliding are provided in Section 5.3.8.

A modified Strength Limit State analysis should be performed that includes in the ice pressures specified in Section 3.9 Ice Loads, with the appropriate strength limit state resistance factors. That Strength Limit State that results in the extreme force and moment effects should be selected.

5.3.3 *Extreme Event Limit States*

Spread footings should be designed for extreme events such as seismic loads, liquefaction, check flood for scour, vessel impact, vehicle or railway collision, and ice.

The ice pressures for the Extreme Event II Limit State should be unfactored and applied at Q1.1 and Q50 elevations as defined in Section 3.9 Ice Loads but with the ice thickness increased by 1 foot.

Resistance factors for extreme event limit states shall be taken as 1.0.

For the extreme event limit state, the Designer should consider scour due to the check flood event and should determine that there is adequate foundation resistance to support all applicable unfactored loads with a resistance factor of 1.0. Flood event loads should include debris loads, where applicable.

Extreme limit state design checks for spread footings shall include checks of:

- Bearing resistance
- Eccentricity
- Sliding
- Overall stability

5.3.4 *Footing Depth*

Footings should be embedded a sufficient depth to provide adequate bearing materials and protection against frost action, erosion and scour.

5.3.4.1 *Bearing Materials*

A footing should ideally be founded on a single material type throughout its bearing length. If a combination of materials is present underlying the footing (i.e., bedrock and granular material) the granular material should be removed to the bedrock surface and replaced with concrete fill. In special situations where constructing a footing on dissimilar materials cannot be avoided, see the Geotechnical Designer.

Footings should be founded on firm soils or bedrock. Any organic, loose, or otherwise unsuitable material encountered at the footing elevation should be removed to the full depth and replaced with compacted granular fill or concrete fill to the bottom of footing elevation. If concrete fill is used under a foundation, the pay limits should be shown as a vertical plane and should be designated as "Pay Limit for Structural Excavation and Concrete Fill". The distance outside the footing for the concrete fill pay limit should be determined for each individual case and must be shown on the design drawings. Foundation bearing conditions should be approved in the field by the Construction Resident or Geotechnical Designer.

5.3.4.2 Footings on Bedrock

For footings supported on bedrock the surface will be cleaned of all weathered bedrock, fractured material, loose soil, and/or ponded water prior to placement of the footing concrete. Smooth bedrock should be roughened or serrated prior to placing concrete to enhance sliding stability. The foundation bearing areas should be approximately level. Bedrock slopes that exceed 4H:1V should be step-serrated or suitably benched to create level steps or a completely level subgrade. For bedrock slopes between 4H:1V and 6H:1V consider dowels into bedrock to control sliding potential.

5.3.4.3 Frost Protection

Footings will be placed below the depth of frost penetration as discussed in Section 5.2.1 Frost. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

5.3.4.4 Scour Protection

Spread footings on soil or erodible rock at stream crossings should be founded at a depth at least 2 feet below scour depth of scour determined for the check flood for scour. Spread footings supported on soil within the stream channel shall be located a minimum of 6 feet below the thalweg of the waterway. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.3.5 *Bearing Resistance*

5.3.5.1 General

Spread footings for abutments and retaining walls are to be proportioned to ensure stability against bearing capacity failure. Safety against deep seated foundation failure shall also be investigated per LRFD 10.6.2.3.

Bearing resistance should be investigated at the strength limit state.

LRFD Article 11.6.3.2 and Figures 11.6.3.2-1 and -2 provide examples for calculating the vertical bearing stress. In general, load factors selected should produce the total extreme force effect. Specific guidance for selection of load factors for bearing resistance is provided in LRFD Figure C11.5.6-1. Where there is a live load surcharge, the factored surcharge force is included over the backfill immediately above the wall base or footing.

Spread footings should be designed such that the factored design stress does not exceed the factored bearing resistance of the soil or rock. The nominal bearing resistance of footings on soil may be estimated using the Munfakh procedure outlined in LRFD Article 10.6.3.1.2. The use of Terzaghi, Meyerhof, or Vesic methods for estimating the nominal bearing resistance is also acceptable. Consideration of shape factors, inclined loads, ground surface slope, and eccentric loading should be included in the calculation, if applicable. A resistance factor shall be applied to the calculated nominal resistance. Structures should be designed such that the maximum factored pressure on the soil or rock under footings does not exceed the factored bearing resistance provided by the Geotechnical Designer.

The bearing resistance at the service limit state will be settlement controlled (typically 1 inch). Presumptive bearing resistance charts based on soil or rock type may be used to determine the service limit state bearing resistance.

For spread footings on bedrock, the design of the footing is typically controlled by overall stability, i.e., failure along discontinuities in the rock mass or eccentricity. Therefore, the Designer should verify overall stability by sizing the footing based on eccentricity at the strength limit state and then checking the nominal bearing resistance at the service and strength limit states.

5.3.5.2 Bearing Stress Distribution

The distribution of soil pressure should be consistent with the foundation material, whether it is soil or bedrock. When proportioning footing dimensions to meet settlement and bearing resistance requirements, the distribution of bearing stress on the effective footing area shall be assumed to be:

- Uniform for footings on soils
- Triangular or trapezoidal for footings on rock

For structural design of footings, a triangular or trapezoidal stress distribution based on factored loads should be used regardless if the footing bears on soil or rock.

When loads are eccentric, the bearing stress is distributed to the effective footing area, $L' \times B'$, where the reduced dimensions are taken as:

- $B' = B - 2e_B$
- $L' = L - 2e_L$

where e_B and e_L are the eccentricities relative to a point at the center of the footing, parallel to the B and L dimensions, respectively.

5.3.5.3 Bearing Resistance Factors

The resistance factors for bearing resistance are provided in Table 5-2.

Table 5-2 Bearing Resistance Factors

Method/Soil/Condition	Bearing Resistance Factor, ϕ_b
Theoretical method (Munfakh et al. 2001) in clay	0.50
Theoretical method (Munfakh et al. 2001) in sand using SPT	0.45
Semi-empirical methods (Meyerhof, 1957, Terzaghi, Vesic) all soils	0.45
Footings on rock	0.45
Plate Load Test	0.50

5.3.6 Settlement

The design of spread footings is frequently controlled by settlement at the service limit state. It is advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme limit states.

Total and differential settlement should be evaluated. The total settlement includes elastic settlement, primary consolidation, and secondary compression. Elastic settlement results from the compression of the material supporting the foundation or from reduction in pore space in nonsaturated soils. Consolidation settlement occurs when saturated, fine-grained soils experience an increase in stress. Some soils, after experiencing primary consolidation settlement, continue to strain after excess pore-water pressures are dissipated. This process is termed secondary compression, or “creep”.

Immediate or elastic settlement should be determined using the Service I Load Combination, specified as unfactored dead load, plus the unfactored component of live loads assumed to extend to the footing level. Time-dependent settlements, i.e., primary consolidation and secondary compression settlement may be determined using the unfactored dead load only. Other factors that can affect settlement, such as embankment loading, lateral and/or eccentric loading, and dynamic or earthquake loads should also be considered, where applicable.

Differential settlement occurs when one load-bearing member of a structure experiences total settlement of a different magnitude than an adjacent load-bearing member. Transportation structures, especially bridges, are not exceptionally tolerant of differential settlements. Deformation limitations will form the upper bound of allowable differential settlements used to design shallow foundations.

5.3.6.1 Tolerable Settlement

Foundation settlement criteria should be consistent with:

- The type of structure
- The function of the structure
- Anticipated service life
- Consequences of unacceptable movement on structure performance
- Long-term maintainability

Tolerable movements are frequently described in terms of angular distortion between members. Angular distortion (δ'/ℓ) between adjacent foundations should be limited to 0.008 radians for simple span bridges and 0.004 radians for continuous span bridges, where δ' is the differential settlement and ℓ is the span length. Angular distortion limits may deviate on a project by project basis, depending on:

- The cost of mitigating settlement through larger foundations, realignment, lightweight fills or surcharge
- Rideability
- Aesthetics
- Safety

Tolerance of the superstructure to lateral movement will depend on the bridge seat or joint widths, bearing type and structure type.

5.3.6.2 Settlement Analyses

Settlement may be estimated using procedures described in LRFD 10.6.2.4 or other generally accepted methods. The soil parameters used shall be based on the results of laboratory or insitu testing, or both. Total and differential settlement should be evaluated.

Settlement of spread footings on sand can be predicted using calculation methods by Hough, Peck-Bazaraa, D'Appolonia, or Schmertmann, as applicable.

5.3.7 Overall Stability

The overall global stability of spread footings on or near an earth slope should be investigated using Service I Load Combination and an appropriate resistance factor. Where a slope supports or contains a structural element, such as a spread footing supporting a wall or abutment, the resistance factor, ϕ , shall be taken as of 0.65

For foundations on spread footings constructed along rivers and streams, scour of foundation materials is evaluated as specified in LRFD 2.6.4.4.2. Extreme limit state design should check that the nominal resistance of the footing and slope remaining after the scour due to the check flood for scour can support the unfactored strength limit state loads with a resistance factor, ϕ , of 1.0

The overall stability of retaining wall spread footings on or near a slope should be evaluated using limiting equilibrium methods of analysis, which employ the Modified Bishop, simplified Janbu, Spencer, or other generally accepted methods of slope stability analysis.

5.3.8 Sliding

Failure by sliding should be investigated for all spread footings bearing on soil or bedrock. Passive earth pressure exerted by fill in front of the footing should be neglected in consideration that the soil may be removed as the result of scour or during future construction, and in consideration that soils in front of the footing will be subject to freeze-thaw weakening over time. If passive pressure is included as part of shear resistance to sliding, consideration should be made to possible removal of the soil in front of the foundation in the future. If passive resistance is included in the resistance, its magnitude is commonly taken as 50% of the maximum passive pressure resistance computed using Rankine Passive resistance. This is the basis of a resistance factor for passive resistance of ϕ_{ep} of 0.50.

The factored resistance against failure by sliding is taken as:

$$R_r = \phi R_n = \phi_s R_f + \phi_{ep} R_{ep}$$

where:

R_n = nominal sliding resistance

ϕ_s = resistance factor for shear resistance between soil and foundation specified in Table 5-3.

R_f = nominal sliding resistance between soil and foundation

ϕ_{ep} = resistance factor for passive resistance = 0.50

R_{ep} = nominal passive resistance of the soil available throughout the design life of the structure.

Table 5-3 Resistance Factors for Sliding of Spread Footings at the Strength Limit State

Soil/Condition	Sliding Resistance Factor, ϕ_s
Precast concrete on sand	0.90
Cast-in-place concrete on sand	0.80
Cast-in-place or precast concrete on clay	0.85
Soil on soil	0.90
Cast-in-place concrete on rock (based on reliability theory analysis of footings on sand)	0.80
Cast-in-place concrete on rock (calibrated to ASD Factor of Safety of 1.5)	0.90

Spread footings should be designed such that the factored resistance to sliding, R_f , is greater than the factored force effects due to the horizontal components of loads. Load factors selected should produce the extreme force effect. The live load surcharge is not included over the heel. Specific guidance for selection of load factors for sliding are provided in LRFD Figure C11.5.6-2.

The nominal sliding resistance between footings and cohesionless soils is taken as:

$$R_f = V \times \tan \delta$$

where:

$$\tan \delta = \tan \phi \text{ for cast-in-place footings on soil}$$

$$\tan \delta = 0.80 \tan \phi \text{ for precast footings on soil}$$

$$V = \text{total vertical force}$$

The coefficient of friction, $\tan \phi$, for sliding should be as shown in Table 3-3 for the soil type under the footing and LRFD Table 3.11.5.3-1.

The nominal sliding resistance between footings and silt and/or clay soils should be taken to be the lesser of: (1) the undrained shear strength of the silt/clay, or, (2) one-half of the normal stress on soil when the footing is founded on at least 6 inches of compacted granular fill on silt/clay.

For footings on bedrock, the Geotechnical Designer will provide a coefficient of friction for sliding. If smooth bedrock is present at the bearing elevation or if the coefficient of sliding is insufficient to resist lateral forces, the bedrock should be doweled to improve stability. When a footing is doweled into rock, the dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the Designer. The spacing of

the dowels should be no greater than 3 feet between rows and no less than two rows. If sloping bedrock is present (steeper than 4H:1V) at the bearing elevation, the bedrock should be benched to create level steps or doweled to improve stability.

5.3.9 *Eccentricity*

Load factors for eccentricity selected should produce the extreme force effect. The live load surcharge is not included over the heel of the footing. Specific guidance for selection of load factors for eccentricity are provided in LRFD Figure C11.5.5-2. The location of the resultant of the reaction forces shall be:

- within the middle two-thirds (2/3) of the footing width or length, B or L, for footings on soils, or
- within the middle nine-tens (9/10) of the footing width or length, B or L, for footings on rock.

5.3.10 *Ground Water Condition*

Footing excavations below the ground water table, particularly in granular soils having relatively high permeability, should be made such that the hydraulic gradient in the excavation bottom is not increased to a magnitude that would cause the foundation soils to loosen or soften due to upward flow of water. Dewatering or cutoff measures to control seepage should be used where necessary. Footing design should be calculated using the highest anticipated ground water level at the footing location.

5.3.11 *Drainage Considerations*

Adequate drainage of materials behind structures is of great importance and should be provided as described in Section 5.4.1.9 Drainage.

5.4 Abutments

5.4.1 *Conventional Abutments*

5.4.1.1 General Design Requirements

Abutment and wingwall design should include evaluation of settlement, lateral displacement, overall stability of the earth slope with the foundation unit, bearing capacity, sliding, loss of contact with foundation soils, eccentricity (overturning), pile capacity (if applicable) and structural capacity. Abutments should be designed for extreme events such as vessel collisions, vehicle collisions, and seismic activities, along with

changed conditions such as scour, as applicable. The design of abutments and walls should satisfy service, strength, and extreme limit state requirements.

5.4.1.2 Loads Combinations and Load Factors

Structural analyses and geotechnical evaluation of abutments should be performed in accordance with the AASHTO LRFD. Abutments should be designed and proportioned to resist all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and as outlined in Chapter 3 Loads.

Abutments should be evaluated for each of the applicable limit states:

- Strength I-construction. Strength Limit State I with the exception that bridge superstructure DC and DW, and vehicular live loads, LL, are neglected. Load factors for the dead load of other components shall not be less than 1.25. Live load surcharge is included to account for construction equipment live loading during structure erection and a construction load factor of not less than 1.5 should be assumed. The Strength I-construction analysis should investigate any anticipated construction loadings, such as looking at the abutment partially backfilled without the superstructure in place.
- Strength I-a: Strength Limit State I, which models the basic load combination related to normal vehicle use of the bridge without wind, dead load plus earth pressure, finished grade, including the vertical component of the superstructure, approach slab, live load effects of traffic on the approach (LS) the vertical component of the live load from superstructure. Minimum vertical permanent load factors and maximum horizontal load factors are selected to produce extreme force effects for abutment sliding and eccentricity, and structural design of the abutment stem.
- Strength 1-b: Strength Limit State 1 as described above, except maximum vertical permanent load factors, including earth loads, are selected to produce an extreme force effect for bearing capacity analyses.
- Strength III: Load combination relating to the bridge exposed to high wind velocity (100 mph) without live loads. Minimum and maximum load factors should be selected for permanent loads to investigate the most extreme force or moment effect.

- Strength IV: Load combination relating to very high dead load to live load force effect ratios exceeding about 7.0 Strength IV will likely govern for bearing failure on long span bridges. It also will likely govern for structural design of the footing. Minimum and maximum load factors should be selected for Permanent Loads to investigate the most extreme force or moment effect.
- Strength V: Load combination relating to the bridge exposed to wind velocity of 55 mph with live loads. Minimum and maximum load factors should be selected for permanent loads to investigate the most extreme force or moment effect.
- Service I: Service Limit State I – Load combination relating to normal operational use of the bridge with a 55 mph wind and all loads taken at their unfactored values.

For the load combinations with all dead loads applied, with or without the superstructure live load, distribute the superstructure loads over the length of the abutment between the fascia lines of the superstructure.

Where abutments are to be designed to resist earthquake forces, collisions by roadway or rail vehicles, or vessel collision, the structures should be evaluated for the following additional limit states:

- Extreme Event I – Load combination including earthquake forces
- Extreme Event II – Load combination relating to collision by vehicles or vessels.

Certain permanent loads, including earth loads, should be factored using the load factors γ_p . Permanent load factors should be selected to produce the total extreme factored force effect. Typical load factors, load combinations and the analyses for which they will govern, are provided in Table 5-4.

Table 5-4 Typical Load Groups and Load Factors (γ_i) for Abutments on Spread Footings

Controlling Load Group	γ_{DC}	γ_{EV}	γ_{Ls}	γ_{EH} (active or passive)	γ_{LL}	Analysis Governed
Strength I-a	0.90	1.0	1.75	1.5	1.75	- Sliding - Eccentricity (overturning) - Structural design of wall stem
Strength I-b	1.25	1.35	1.75	1.5	1.75	- Bearing Capacity
Strength IV	1.50	1.35	--	1.5	--	- Bearing capacity - Structural design of the footing
Service I	1.0	1.0	1.0	1.0	1.0	- Settlement - Lateral movement - Angular distortion

Longitudinal forces for abutment design should include any live load longitudinal forces developed through bearings such as braking forces, or others as specified in LRFD Article 3.0, unless limited by friction capacity.

5.4.1.3 General

The Designer should estimate the load combinations which could be imposed on the abutment or wall and estimate the nominal resistance of the structural component or ground. Abutment components shall satisfy the following equation for each limit state:

$$\sum \eta_i \gamma_i Q_i \leq \Phi R_n = R_f$$

where:

η_i = Factors to account for ductility, redundancy and operational importance

γ_i = Load factor (dim)

Q_i = Load or stress

Φ = Resistance factor (dim)

R_n = Nominal resistance

R_f = Factored resistance

5.4.1.4 Strength Limit State Evaluations

The above equation should be used to evaluate abutments at the strength limit states for:

- Bearing resistance failure
- Lateral sliding
- Excessive loss of base contact (eccentricity)
- Pile failure
- Structural failure

The factored resistance, R_f , calculated for each mode of failure, is to be calculated using the appropriate resistance factors for bearing resistance, sliding, eccentricity, axial pile resistance and structural resistance.

The Designer should consider the consequences of changes in abutment foundation conditions at the strength limit state resulting from scour due to the design flood event using appropriate resistance factors.

5.4.1.5 Service Limit State Evaluations

Abutments should be investigated at the service limit state using the load and resistance equation in Section 5.4.1.3 for:

- Settlement
- Lateral displacement
- Overall slope stability
- Overall stability at the design flood

A resistance factor, ϕ , of 1.0 is used to assess abutment design at the service limit state. Overall stability of abutments on or near earth slopes should be investigated using resistance factors in Section 5.3.7 Overall Stability.

Tolerable vertical and lateral displacement criteria for abutment shall be developed based on the function and type of wall, anticipated service life, and consequences of unacceptable movements of the wall and effect on nearby structures. To control bridge superstructure damage, a limiting horizontal movement of abutments less than 1.5 inch is recommended. Utilities may not be able to accommodate very large movements, in which case a project-specific limiting movement should be developed.

5.4.1.6 Extreme Limit State Evaluations

Extreme limit state design checks for abutments should include:

- Bearing resistance
- Eccentricity
- Sliding
- Overall stability

A resistance factor, ϕ , of 1.0 is used in the load and resistance equation in Section 5.4.1.3 to assess abutment design at the extreme limit state.

The extreme event limit state design should check that the nominal abutment foundation resistance after scour due to the check flood event can support all applicable unfactored loads with a resistance factor of 1.0. For abutments on spread footings, refer to 5.3.4.4. For pile-supported abutments, refer to 5.4.1.12.

5.4.1.7 Load Considerations

A. Earth Loads

For abutment and wingwall designs, use the appropriate soil weight shown for Soil Type 4 (Table 3-3) for soil properties for backfill material. Abutments and retaining walls should be designed as unrestrained and free to rotate at the top in an active state of earth pressure. An active earth pressure coefficient, K_a , should be calculated using Rankine Theory for long-heeled cantilever abutments and wingwalls, and Coulomb Theory for short heeled cantilever abutments and gravity shaped walls. Refer to Section 3.6.5.1 Coulomb Theory. Soil Type 4 properties are consistent with materials typically used for backfill behind abutments and retaining walls. For unconventional backfills, i.e., tire shreds, light weight fills, etc., consult the Geotechnical Designer or Report.

B. Unit Weight of Concrete

A unit weight of 150 lb/ft³ should be used for design purposes.

C. Live Load Surcharge Loads

Abutments without approach slabs should be designed with a live load surcharge when computing horizontal earth pressure. This additional lateral pressure on walls is approximated by a uniform horizontal earth pressure due to an equivalent height of soil, H_{eq} . Refer to Section 3.6.8

Surcharge Loads for guidance in computing this additional lateral surcharge pressure.

Wingwalls and retaining walls should also be designed for surcharge loads in accordance with Section 3.6.8.

In the case a structural approach slab is specified, reduction, but not elimination, of the surcharge loads is permitted per LRFD 3.11.6.2.

D. Lateral Loads

Load conditions should include any additional lateral pressures on the walls. These loads may include but are not limited to impact loads transmitted to the retaining walls from distribution slabs supporting crash barriers.

E. Collision Forces

Unless the department determines that site conditions indicate otherwise, abutments within a distance of 30 feet to the edge of a roadway or within 50 feet to the centerline of railway track shall be investigated for collision. Collision loads and crashworthy barrier design criteria for abutments are identical to those provided for Piers in Section 5.5.1.10 Pier Protection.

5.4.1.8 Backfill

Abutment walls and footings should be backfilled with granular borrow for underwater backfill. Extend underwater granular backfill for a horizontal distance of at least 10 feet from the back face of the abutment wall and 1 foot behind the back face of the footings.

5.4.1.9 Drainage

The Designer should study total drainage design. Adequate drainage of fill behind structures is important to increase the longevity of retaining structures. Water should not drain into the underside of slope protection. Drainage should be provided as follows:

- Where possible, french drains should be used at the back face of walls with 4 inch diameter drain pipes (weep holes) at nominal 10 foot maximum spacing through the walls . Refer to Standard Specification Section 512 – French Drains.
- Underdrains or other means may be used where necessary to provide adequate drainage.

5.4.1.10 Reinforcement and Structural Design

The structural design of abutments should comply with the requirements of AASHTO LRFD. Earth loads for structural design should be calculated per Section 3.4, Earth Loads, and an appropriate load factor applied.

Concrete cover for footing reinforcement should be as specified by AASHTO LRFD, except that for "non-designed" footings, such as for stub abutments, 6 inches of cover should be used.

At the back corners of gravity abutments and wingwalls, horizontal rebar should be placed, #6 bars at 12 inches on center, with lengths of 8 feet and with 6 inches of cover. Also, four #6 bars, 8 feet long, should be placed at 6 inches below bridge seat elevation at the front corners.

5.4.1.11 Abutments on Spread Footings

A. General

Refer to Section 5.3 Spread Footings for guidance on the design of spread footings.

The general design process for spread footing design should follow the steps below:

1. Determine the nominal and factored footing resistances at the service, strength and extreme limit states assuming footing dimensions and depth (consult Geotechnical Design Report)
2. Determine the loads applied to the footing, including lateral earth pressure loads for the abutment
3. Initially size and design the footing at the service limit state
4. Check the bearing pressure of the footing at the strength limit state
5. Check the eccentricity of the footing at the strength limit state
6. Check the sliding resistance of the footing at the strength limit state
7. Check the bearing pressure and eccentricity and sliding resistance of the footing at the extreme limit state
8. Check the footing bearing resistance at all limit states and overall stability in light any refined/new footing dimensions, depth and loads provided by the Designer.

9. Reassess steps 4 thru 7 based on the revised nominal and factored footing bearing resistance calculated

B. Spread Footings on Bedrock

Refer to Section 5.3.4.2 for guidance on the design of spread footings on bedrock.

C. Vertical and Horizontal Displacement

Vertical and horizontal movement criteria for abutments should be developed consistent with the function and type of structure, consequences of unacceptable movements on structure performance and the cost of mitigating movements and/or rotations by larger foundations. Angular distortions and settlements should be designed per Section 5.3.6 Settlement.

D. Global Stability

Global stability of slopes with abutments or walls should be considered part of the design of the wall or abutment. Evaluation of the global stability of an abutment is important when the abutment is located close to or on an inclined slope, or close to an embankment, excavation, or retaining wall.

The evaluation of the overall stability of earth or rock slopes with walls and abutments shall be investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65. Refer to 5.3.7 Overall Stability for additional guidance.

E. Bearing Stress

Maximum bearing stress under footings at the strength limit load combination should be determined per Section 5.3.5 Bearing Resistance. Structures should be designed such that the calculated factored bearing stress under footings does not exceed the factored soil or rock bearing resistance in accordance with recommendations of the Geotechnical Designer. This requirement is expressed below:

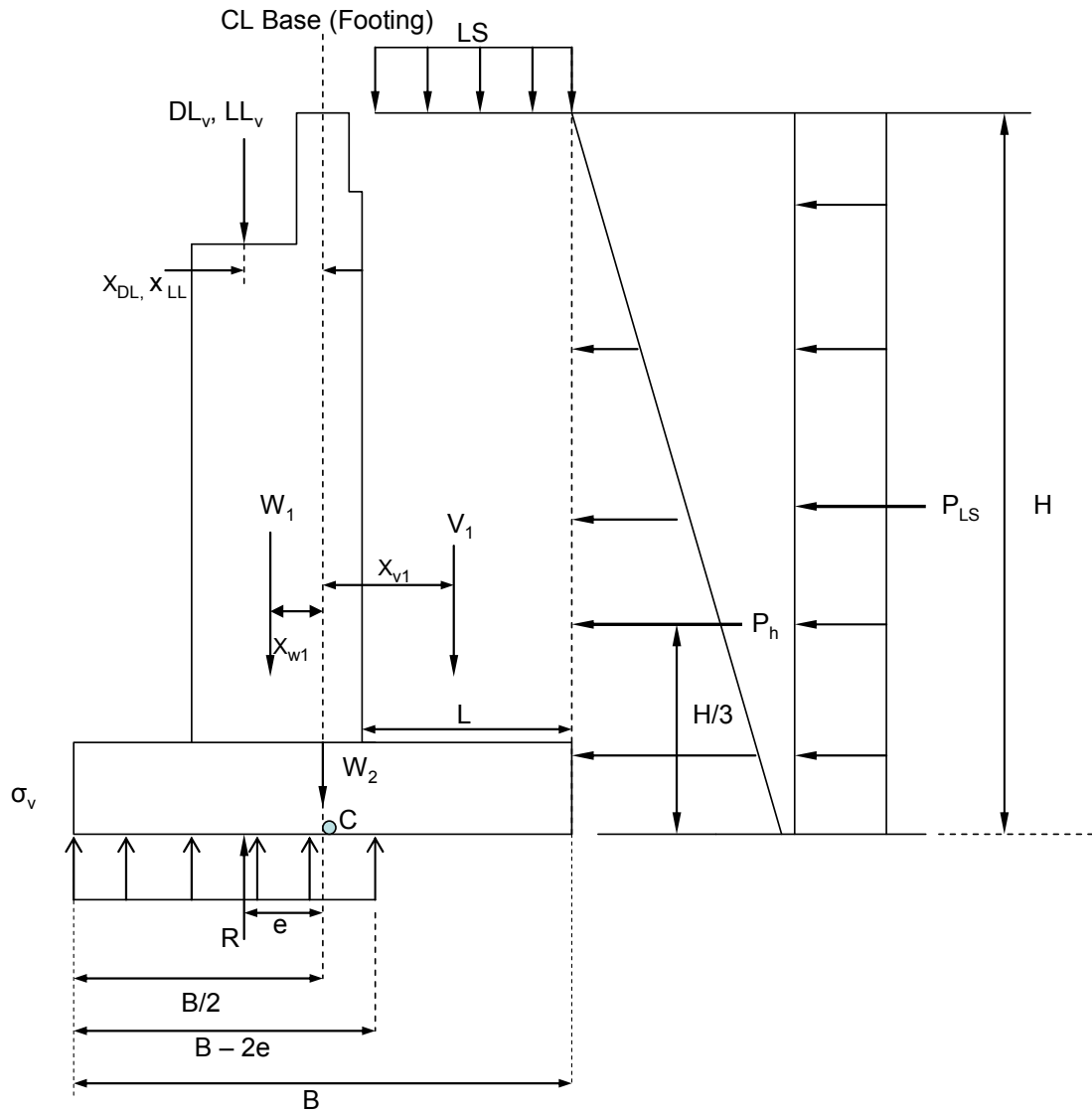
$$\sigma \leq \phi R_n = R_f$$

where:

- σ = factored vertical stress (ksf)
- ϕ = bearing resistance factor (dim)
- R_n = nominal bearing resistance (ksf)
- R_f = factored bearing resistance (ksf)

The weight of the earth in front of a wall should be considered in computing maximum bearing pressure. When loads are eccentric, the effective footing dimension should be used for the overall dimension in the equation for bearing resistance. Refer to Procedure 5-1 and Procedure 5-2 for how to calculate applied bearing stress.

**Procedure 5-1 Bearing Stress on Soil
For Wall or Conventional Abutment**



Step 1. Calculate eccentricity, e_c , about point C, where:

M_o = sum of moments of factored overturning forces acting about point C:

$$M_o = P_h \cdot \frac{H}{3} + W_1 \cdot x_{w1} + P_{LS,h} \cdot \frac{H}{2} + DL_v \cdot x_{DL} + LL_v \cdot x_{LL}$$

M_r = sum of moments of factored resisting forces acting about Point C:

$$M_r = V_1 \cdot x_{v1} + LS \cdot L \cdot x_{v1}$$

ΣV = sum of factored vertical forces acting on the footing and wall:

$$\Sigma V = V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L$$

and,

$$e_c = \frac{M_o - M_r}{\Sigma V}$$

$$e_c = \frac{P_H \cdot \frac{H}{3} - V_1 \cdot x_{v1} - LS \cdot L \cdot x_{v1} + W_1 \cdot x_{w1} + P_{LS,h} \cdot \frac{H}{2} + DL_v \cdot x_{DL} + LL_v \cdot x_{LL}}{V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L}$$

Step 2. The factored vertical stress should be calculated assuming a uniformly distributed pressure over an effective base area shown in the Figure above. The vertical stress should be calculated as follows:

$$\sigma_v = \frac{\Sigma V}{B - 2e_c}$$

Note that B-2ec is considered to be the effective footing width.

Step 3: Compare σ_v which already has the load factors included, to the factored bearing resistance of the soil, provided in the Geotechnical Report. The maximum factored stress should be less than the factored bearing resistance.

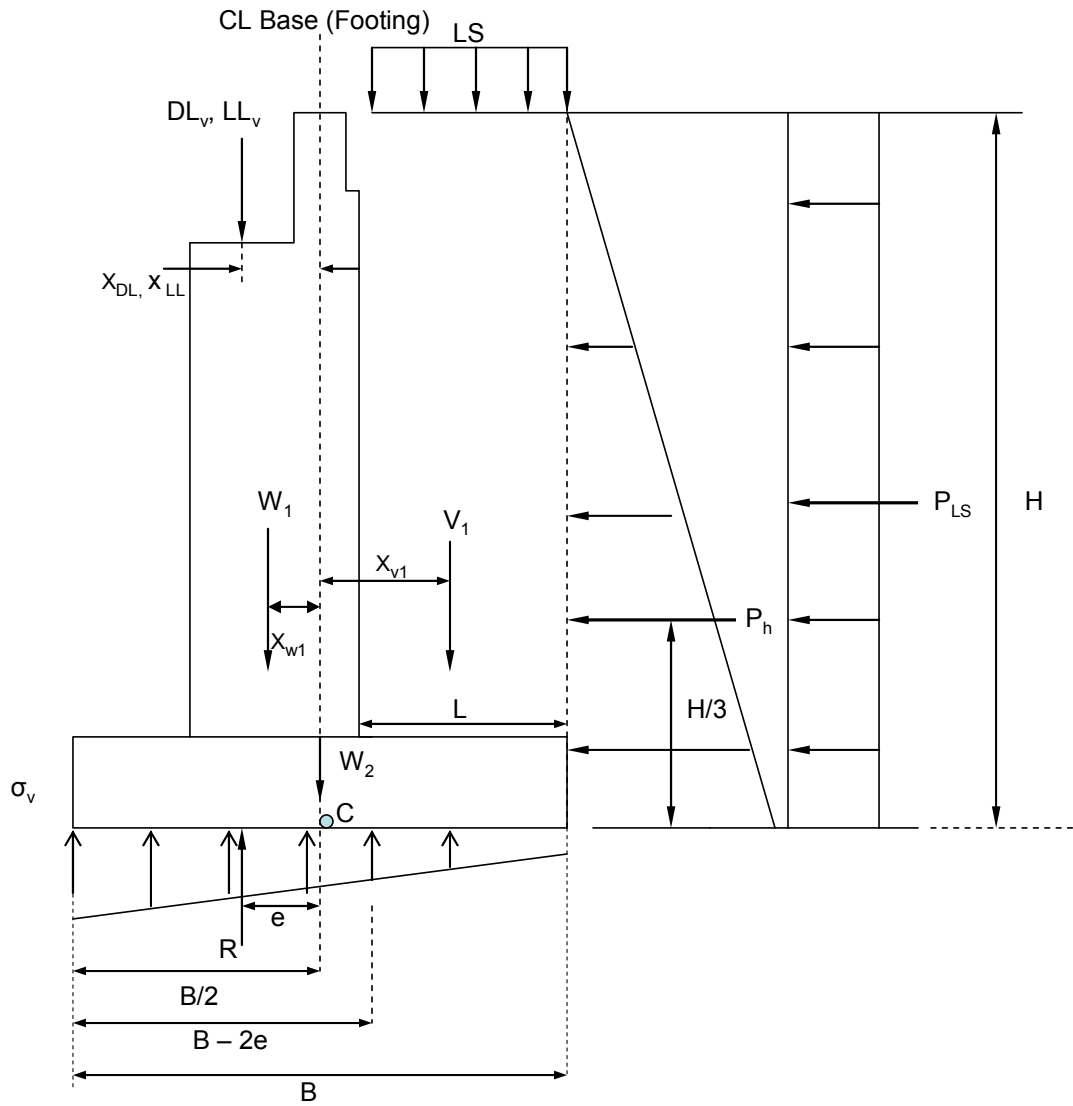
$$\sigma_v \leq \phi_{bc} \cdot R_n = R_f$$

where:

- σ_v = factored vertical stress (ksf)
- ϕ_{bc} = bearing resistance factor (dim)
- R_n = nominal bearing resistance (ksf)
- R_f = factored bearing resistance (ksf)

Note: The case shown for this procedure is the construction load with full backfill and live load surcharge on the approach, and superstructure dead load. For other load combinations, the appropriate loads must be included in the analysis.

**Procedure 5-2 Bearing Stress on Bedrock
For Conventional Abutment**



Step 1: Calculate the eccentricity about point C, e_c , where:

M_o = sum of moments of factored overturning forces, acting about point C:

$$M_o = P_h \cdot \frac{H}{3} + W_1 \cdot X_{w1} + P_{LS,h} \cdot \frac{H}{2} + DL_v \cdot X_{DL} + LL_v \cdot X_{LL}$$

M_r = sum of moments of factored resisting forces about Point C:

$$M_r = V_1 \cdot X_{V1} + LS \cdot L \cdot X_{LS}$$

ΣV = sum of factored vertical forces acting on the footing and wall:

$$\Sigma V = V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L$$

and,

$$e_c = \frac{M_o - M_r}{\Sigma V}$$

$$e_c = \frac{P_H \cdot \frac{H}{3} - V_1 \cdot X_{V1} - LS \cdot L \cdot X_{LS} + W_1 \cdot X_{W1} + P_{LS,h} \cdot \frac{H}{2}}{V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L}$$

Step 2: The factored vertical stress should be calculated assuming a linearly distributed pressure over an effective base area shown in the figure above. If the resultant is within the middle 1/3 of the base, the maximum and minimum factored vertical stress is calculated as follows:

$$\sigma_{v \max} = \frac{\Sigma V}{B} \cdot \left(1 + 6 \cdot \frac{e_c}{B} \right)$$

$$\sigma_{v \min} = \frac{\Sigma V}{B} \cdot \left(1 - 6 \cdot \frac{e_c}{B} \right)$$

If the resultant is outside of the middle 1/3, of the base, i.e., if $B/6$, $\sigma_{v \min}$ will drop to zero, and as “e” increases, the portion of the heel of the footing which has zero vertical stress increases.

$$\sigma_{v \max} = \frac{2 \cdot \Sigma V}{3 \cdot \left(\frac{B}{2} - e_c \right)}$$

$$\sigma_{v \min} = 0$$

Step 3: Compare $\sigma_{v \max}$ to the factored bearing resistance, q_r , provided in the Geotechnical Report. The maximum factored bearing stress should be less than the factored bearing resistance.

$$\sigma_{v \max} \leq \phi_{bc} \cdot R_n = R_f$$

where:

$\sigma_{v \max}$ = maximum factored vertical stress (ksf)

ϕ_{bc} = bearing resistance factor (dim)

R_n = nominal bearing resistance (ksf)

R_f = factored bearing resistance (ksf)

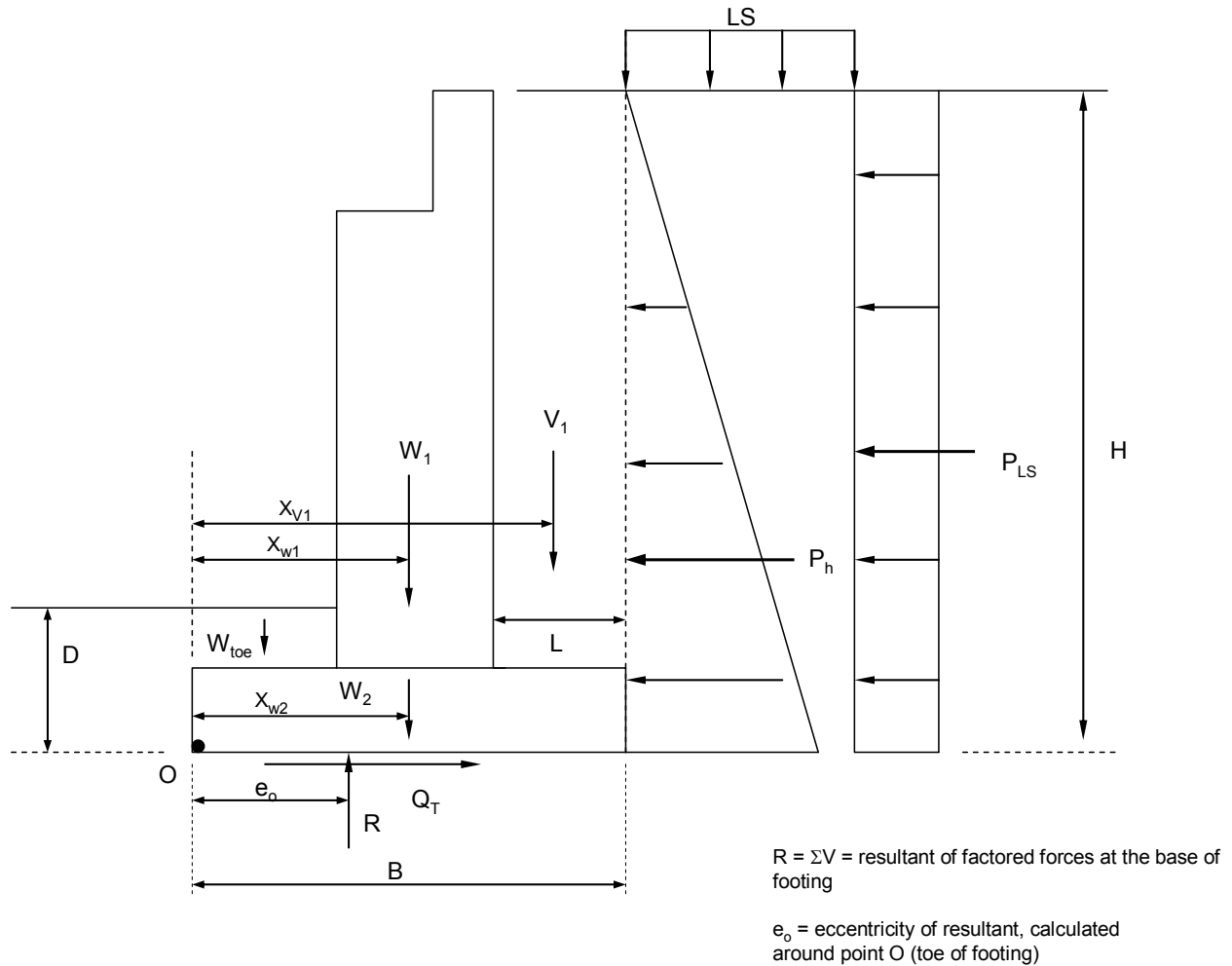
Note: The case shown for this procedure is the construction load with full backfill and live load surcharge on the approach. For other load combinations the appropriate loads must be included in the analysis.

F. Sliding

Failure by sliding should be investigated for all abutments founded on spread footings bearing on soil or bedrock. Passive earth pressure exerted by fill in front of the footing is neglected in consideration that soil may be removed during future construction. Refer to Section 3.6.9 Passive Earth Pressure Loads for guidance. The factored resistance against failure by sliding of abutments and walls on spread footings shall be calculated as described in Section 5.3.8 and LRFD 10.6.3.4. Resistance factors for sliding of spread footings at the strength limit state are provided in Table 5-3.

The coefficient of friction for sliding should be as shown in Table 3-3 for the appropriate soil type under the footing. For footings on bedrock, the Geotechnical Designer will provide a coefficient of friction for sliding, based upon the bedrock characteristics.

Procedure 5-3 Eccentricity and Sliding Check for Conventional Abutment on Spread Footing



Step 1: Calculate the eccentricity about Point O in the Figure above to locate the resultant force R. Forces and moments resisting overturning are to be considered negative, and the maximum load factors should be used (Table 5-4)

$M_o =$ sum of moments of factored overturning forces acting about Point O:

$$M_o = P_h \cdot \frac{H}{3} + P_{LS} \cdot \frac{H}{2}$$

$M_r =$ sum of moments of factored resisting forces acting about Point O:

$$M_r = V_1 \cdot x_{V1} + W_1 \cdot x_{w1} + W_2 \cdot s_{w2}$$

$\Sigma V =$ sum of factored vertical forces acting on footing and wall, as defined in the Figure above.

$$\sum V = V_1 + W_1 + W_2$$

Step 1: Check eccentricity (overturning) about Point O:

$$e_o = \frac{M_o - M_r}{\sum V}$$

For footings on soil, the location of the resultant force shall be within the middle two-thirds (2/3) of the base width. For footings on bedrock, the location of the resultant force shall be within the middle nine-tenths (9/10) of the base width. For footings subjected to biaxial loading, these eccentricity requirements apply in both directions.

Step 2: Compare the factored resistance to sliding to the factored applied horizontal loads. The factored resistance to sliding should be greater than the factored applied horizontal loads:

$$R_n = \sum V \cdot \tan \delta$$

$$R_f = R_n \cdot \phi_s > Q$$

where:

R_n = Nominal sliding resistance between soil and foundation (kips)

δ = friction angle between the footing base and the soil (refer to Table 3-3 or LRFD Table 3.11.5.3-1.)

ϕ_s = resistance factor for shear resistance between the soil (or rock) and foundation

Q = factored horizontal applied loads

Note: The load combination shown for this strength limit state is Strength I-a, which does not consider superstructure dead loads (DC and DW) and vehicular live loads (LL). For other load combinations the appropriate loads and load factors must be included in the analysis.

G. Eccentricity

Abutments and walls on spread footings should be designed to resist overturning which results from lateral and eccentric vertical loads. The eccentricity should be evaluated as shown in Procedure 5-3. The location of the resultant of the reaction forces of at the strength limit state, based on factored loads, shall be within the middle two-thirds (2/3) of the footing width for footings on soil or the middle nine-tenths (9/10) of the footing width for footings on rock.

If construction loading is critical, the backfill height may be restricted until the superstructure or other parts are constructed.

5.4.1.12 Abutments Supported on Pile Foundations

Piles should be designed in accordance with the requirements of Section 5.7 Piles.

For pile supported abutments, the factored load combination causing the maximum and minimum compression in the piles should be determined, and the resulting pile reactions and pile stresses determined. The maximum factored axial pile load should not exceed the lesser of the factored geotechnical resistance and factored structural resistance for a single pile. In accordance with LRFD Article 6.5.4.2, the factored pile loads should not exceed the factored structural resistance using the resistance factors provided in 5.7.2 H-Piles and 5.7.5 Steel Pipe Piles. If greater loads result, more piles, or larger piles, should be considered.

For the Service Limit State, the unfactored lateral pile loads for H-piles should not exceed the lateral loads resistances specified in 5.7.2.2

Load combinations that do exceed the lateral load limits established for the service limit state should be evaluated by the Geotechnical Designer by means of a project-specific pile lateral load analysis using LPILE[®] software. The maximum lateral loads for all piles other than steel H-piles should be evaluated by the Geotechnical Designer. Buckling analyses of piles should be performed by the Structural Designer. Piles should also be checked for resistance against combined axial loads and flexure per LRFD 6.15 and 6.9.2.2. Pile resistance should be determined for compliance with the LRFD interaction equation.

Where abutments are required in water channels, the bottom of seal should be a minimum of 2 feet below the calculated scour depth from the check flood for scour. Where the calculated scour depth is significant, the Designer may consider designing the deep foundation elements for an unsupported length. The unsupported length should be the vertical distance from the bottom of the seal to the check flood scour depth. In designing deep foundation elements for an abutment with an unsupported length, a complete analysis of the foundation should be performed using actual loading and soil conditions.

Vertical and horizontal movement criteria for abutments supported by pile foundations should be developed consistent with the function and type of structure. The effect of lateral squeeze in the pile-supported abutments should be considered by the Geotechnical Designer, if applicable. Refer to Sanford, October 1994.

5.4.1.13 Bridge Seat Dimensions

As a minimum, the bridge seat dimensions should meet the requirements of LRFD Article 4.7.4.4. Otherwise, for bridge seats supporting steel superstructures exceeding 200 feet, use a minimum of 2 feet between the centerline of bearings and the face of breastwall and a minimum of 2'-3" between the centerline of bearings and the face of backwall. The masonry plate of the bearings should be no closer to the face of breastwall than 3 inches and should clear the face of backwall by at least 2 inches. For steel superstructures between 100 and 200 feet use a minimum 3 foot bridge seat. For steel superstructures less than 100 feet, the bridge seat dimensions should be large enough to accommodate the bearing masonry plate and the previous clearance dimensions. For major steel structures, all precast concrete structures, and structures with skews exceeding 45°, the bridge seat dimension should be determined based upon the project requirements.

All bridge seats, regardless if protected from roadway drainage by sealed bridge joints, should be concrete pedestal type with a minimum width along the centerline of bearing of 3 feet. The clear distance between the ends of bearing masonry plates and the ends of concrete pedestals should be at least 6 inches. The bridge seat between concrete pedestals should be sloped downward toward the face of breastwall at a slope of at least 15%.

Top of abutment backwalls should be 1'-6" wide, excluding the 6 inch approach slab seat, except when the concrete superstructure slab extends over the top of the backwall and the back of the backwall is battered. In that case, the backwall should be 1'-6" plus the effect of the batter.

5.4.2 Integral Abutments

5.4.2.1 Introduction

There are two categories of integral abutments: (1) full integral abutments, where the bridge beams are rigidly cast into an end diaphragm and (2) integral with hinge abutments, where butted boxes or voided slabs are connected to the abutment with dowels.

Integral abutment bridges (IABs) should be evaluated for use on all bridge replacement projects. MaineDOT most commonly uses 4 piles for each integral abutment substructure unit and traditionally uses the following piles:

- HP 10x42
- HP 12x53

- HP 14x73
- HP 14x89

Design is not limited to these piles. If the Structural Designer elects to use a pile not listed, the appropriate design analysis must be conducted.

Although HP 14 x 73 pile flanges are non-compact and do not meet the slenderness requirements of LRFD 6.9.4.2, Designers can account for pile slenderness in the design process, and this pile size should still be considered for pile supported integral abutments.

5.4.2.2 Loads

Analysis and design of integral abutment substructures will be in accordance with AASHTO LRFD, and include structural design and analysis of reinforced concrete abutments and wings, global stability of the channel slope with abutment, and pile design. Load combinations are presented in Section 5.4.1.2. Additional appropriate load combinations that investigate the effects of thermal gradients and abutment displacement may be required in accordance with LRFD Section 3.

5.4.2.3 Historical ASD Design Practice and Bridge Lengths

Commentary: Design of integral abutment bridges has evolved over the years as transportation departments have gained confidence with the system. Bridge lengths have gradually increased without a rational design approach. Tennessee, South Dakota, Missouri and several other states allow lengths in excess of 300 feet for steel structures and 600 feet for concrete structures.

Thermally-induced pile head translations in bridges with the lengths stated above will cause pile stresses which exceed the yield point. Research performed during the 1980's (Greimann, et. al.) resulted in a rational design method for integral abutment piles, which considers the inelastic redistribution of these thermally induced moments. This method is based upon the ability of steel piles to develop plastic hinges and undergo inelastic rotation without local buckling failure. This method is not recommended for concrete or timber piles, which have insufficient ductility.

Past practice was based on evaluation of the four steel piles most commonly used by MaineDOT and maximum bridge lengths and maximum design pile load design guides were developed based upon the Greimann research. The pile were evaluated as beam-columns without transverse loads between their ends, fixed at some depth and either pinned or fixed at their heads.

Greimann, et. al., developed design criteria by which the rotational demand placed upon the pile must not exceed the pile's inelastic rotational capacity. The following system variables affect the demand:

- Soil type
- Depth of overlying gravel layer
- Pile size
- Pile head fixity
- Skew
- Live load girder rotation

In order to simplify the design, past practice assumed that piles would be driven through a minimum of 10 feet of dense gravel. Material below this level has very little influence on pile column action. It was also assumed that the live load girder end rotation stresses induced in the pile head do not exceed $0.55 F_y$ (which provides a known live load rotational demand). Based upon the above assumptions and the pile's inelastic rotational capacity, the maximum pile head translation, Δ (in inches) was established for each of the four piles. Based on allowable stress design, the maximum bridge lengths historically were as follows:

- $MaxBridgeLength \cdot ft = \frac{4 \cdot \Delta \cdot in}{0.0125}$ for steel bridges
- $MaxBridgeLength \cdot ft = \frac{4 \cdot \Delta \cdot in}{0.075}$ for concrete bridges

Maximum bridge lengths vary from 70 feet to 500 feet for some piles. The past practice for maximum bridge lengths was 200 feet for steel and 330 feet for concrete. FHWA allows maximum bridge lengths of 300 feet for steel bridges, 500 feet for cast-in-place concrete bridges, and 600 feet for prestressed or post tensioned concrete bridges (FHWA Technical Advisory, January 28, 1990). Refer to BDG 5.4.2.6 for current bridge length limits.

5.4.2.4 Pile Design

A. Pile Loads

Piles should be modeled and evaluated as either fixed at the pile head for fully integral abutments (bridge beams are rigidly cast into an end diaphragm) or as pinned for integral abutments with hinge, such as the case when butted boxes or voided slabs have dowel connections to the abutment.

Piles for full integral and integral with hinge abutments shall be designed to resist all vertical superstructure dead and live loads, abutment and pile dead loads, live load girder rotation moments, lateral displacements, live load impact and moments caused by superimposed dead loads and live loads, as appropriate for the type of integral abutment.

Until the behavior of integral abutments with hinged connections to the superstructure is better understood, the pile design criteria for that type of integral abutment may assume that the moment at the top of the pile is zero, and that there is no moment from either the superstructure or earth loads.

The effect of thermal displacements and moments on piles can be investigated by running LPILE[®] software.

Secondary thermal forces only need be considered for multi-span structures only.

Appropriate load combinations and load factors should be determined per LRFD 3.4.1.

For the strength limit state analysis, design of the piles should consider the factored structural pile resistance, P_r , the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.

For service limit state evaluations, if piles will be driven to practical refusal in bedrock, settlement will not be a concern. However, all designs should consider horizontal movement, overall stability and scour for the design flood event.

B. Resistance Factors for Integral H-Piles

Pile will typically be end bearing on bedrock. For the strength limit state, use the following resistance factors:

- Use $\Phi_c = 0.50$ for axial resistance in compression and subject to severe pile driving condition; this condition should be assumed when analyzing the lower portions of the pile
- Use $\Phi_c = 0.60$ for axial resistance in compression under good driving conditions; this condition should be assumed when analyzing the upper portion of the pile
- For combined axial and flexural resistance in the upper zone of pile, use:

- $\Phi_c = 0.70$ for axial resistance
- $\Phi_f = 1.00$ for flexural resistance

C. Design Steps

The following steps should be followed during design of piles supporting full integral abutments, for the strength limit state:

1. Determine the foundation displacements, and the load effects (P_u and M_u) from the superstructure and substructure designs.
2. If applicable, determine the magnitude of scour.
3. Select preliminary pile size:
 - a. Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile
 - b. Select the steel pile strength
 - c. Select pile orientation; typically weak axis bending
 - d. Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.
 - e. Determine the maximum, required nominal axial pile resistance, P_u/Φ_f
 - f. Estimate an initial pile area using the approximation

$$A_s = \frac{Ru}{0.80 \times F_y}$$

This approximation is based on weak axis bending and an assumed unbraced length of 15 feet based on typical integral abutment pile deflection and moment with depth curves. Select a pile size with an area A_s or greater.
4. Determine the pile unbraced length and maximum moment at the top of the pile by running LPILE[®] software for the design displacement from Step 1, P_u , and live load rotation
5. Determine if the applied moment on the pile will cause pile head plastic deformation by using the Interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)
 - a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment. (LRFD 6.9.4.1)
 - b. Determine K values for the top and bottom of the pile per LRFD Table C4.6.2.5-1

- g. Calculate the nominal and factored structural pile resistance P_n , per LRFD 6.9.4.1 using the λ values
 - h. Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.
 - i. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)
 - j. Calculate the moment that will cause a plastic hinge at the top of the pile (M_p')
 - k. If the applied moment exceeds the moment that would cause a plastic hinge, a plastic hinge forms, and the moment that can be applied cannot exceed that moment (M_p')
6. For fixed head piles, run a second LPILE[®] analysis with displacement and plastic moment (M_p') as load conditions and P_u , and calculate new unbraced lengths from the moment with depth curve.
- a. Repeat steps 5.a. through 5.d., above
 - b. If the pile size is such that the ratio of P_u to structural resistance exceeds 0.2, check the upper zone of the pile with the interaction equation of LRFD 6.9.2.2. If a plastic hinge forms at the top of the pile, the K value of the upper segment (that portion between the top of the pile and the first inflection point on the moment vs. depth curve) changes from 1.2, for a pinned condition, to 2.1, for a free condition at the top. With the new K value repeat Step 5, and check the interaction equation for pile overstress.
7. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPILE[®] output to the structural shear resistance per AISC G7.
8. Check that the maximum factored applied pile load does not exceed the factored geotechnical pile resistance or pile drivability resistance (LRFD 10.5.5.2.3 and 10.7.3.13) provided in the Geotechnical Design Report.

5.4.2.5 Pile Length Requirement

A. General Requirements

Piles may be end bearing or friction piles. In order to obtain the pile behavior associated with the equivalent length, piles should be installed 1 to 5 feet beyond the pile length required to achieve fixity. The practical

depth to pile fixity is defined as the depth along the pile to the point of zero lateral deflection.

A minimum pile length of 10 feet is recommended, however soil conditions and loading conditions may require additional pile embedment to achieve fixity. Additional embedment length may be required for the use of friction piles. Also, axial loads may govern and additional embedment length may be required in order to achieve the factored design axial load with appropriate resistance factor applied. For pile lengths less than 14 feet, consideration should be given to the pile translating as a column and the pile tip walking. More vigorous driving shoes designed to properly seat piles and hold the pile and point in place are available. Refer to paragraph B. Short Pile Usage Guidelines, below.

If site-specific soil properties and loading conditions exist, an evaluation of minimum embedment length to achieve fixity using LPILE[®] software or the Davisson and Robinson equation in LRFD 10.7.4.2 is recommended. Consult the Geotechnical Designer for these analyses.

Piles should be driven with their weak axis perpendicular to the centerline of the beams, regardless of skew. Refer to Section 5.7.2 H-Piles for additional design requirements.

When scour is anticipated, the minimum pile length should be provided beyond the depth of computed scour.

B. Short Pile Usage Guidelines

The MaineDOT and the University of Maine at Orono (UMaine) have investigated the performance of integral abutment bridges at sites with shallow bedrock and have instrumented and monitored Nash Stream Bridge in Coplin Plantation, Maine, (Hartt, et. al., 2006 and Delano, et. al., 2005)). Preliminary evaluation of the field data from the research study indicate that integral abutment bridges with 'short' steel piles (14 feet or less) may not develop fixity but perform adequately and do not experience stresses larger than those seen by longer piles. The shortest pile instrumented by the researchers was a 14-foot long H-pile.

To accommodate integral abutment piles at sites with shallow bedrock, the following design features are recommended:

- In consideration of (a) the consequences of scour and pile exposure, (b) the need to limit pile tip movement, and (c) obtaining pile behavior associated plastic stress redistribution and inelastic rotation in the pile, a minimum pile length of 10 feet is recommended. This recommendation is based on finite element analyses and limited field data from the UMaine studies (Delano,

et. al. 2005 and Hartt, et. al. 2006). If the depth to bedrock is so shallow that 10 feet of embedment in soil cannot be achieved, piles should be installed in bedrock sockets to provide the minimum 10-foot pile length recommended. If a fixed condition at the pile tip is desired, the bottom 6-inches of the rock sockets should be tremie-filled with concrete. However, the UMaine research indicates some rotation at the pile tip is acceptable.

- Short piles supporting integral abutments should be designed in accordance AASHTO LRFD criteria and checked for pile tip movement by conducting a LPILE[®] analysis, or as described in the design example found in Appendix B of Technical Report ME 01-7 (Delano, et. al. 2005), and Chapter 5 of that report. Achievement of an assumed pinned condition at the pile tip should also be confirmed with an LPILE[®] analysis.
- Since the abutment piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribe in LRFD Articles 6.9.2.2 and 6.15.2 and checked for compliance with the interaction equation. An LPILE[®] analysis is recommended to evaluate the soil-pile interaction with factored axial loads, moments and pile head displacements applied.
- Driven piles should be fitted with special driving points to improve penetration into bedrock and improve friction at the pile tip to support a pinned pile tip assumption.
- The stream velocity should be low and there should be low potential for removal of any dams, scour action, wave action, storm surge and ice damage. This is to ensure the long-term integrity of the bridge approach fills and riprap abutment slopes, which provide the only lateral support to pile groups.
- Minimum 1.75H:1V slopes in front of integral abutment pile groups should be protected with riprap over an erosion control geotextile or concrete slope protection.

5.4.2.6 Maximum Bridge Lengths

The criteria for the maximum bridge lengths provided in Table 5-5 are based on the following assumptions:

- Steel H-piles are used with their webs oriented normal to the centerline of the bridge (longitudinal translation about the weak axis).

- The piles are driven through gravels or through clays with a minimum of 10 feet of gravel overburden.
- For skews greater than 20°, abutment heights are <12 feet and pile spacing is < 10 feet.
- Total thermal movement is 1-1/4"/100 feet bridge length for steel structures and 3/4"/100 feet bridge length for concrete structures (FHWA Technical Advisory, January 28, 1990).
- Factored pile loads do not exceed the factored compressive structural pile resistance, the factored flexural pile strength and the factored geotechnical and drivability resistance of the pile section.
- Steel H-Piles are made of Grade 50 steel.

Bridge lengths in excess of the limitations below may be used with the approval of the Engineer of Design when special design features are provided.

Table 5-5 Recommended Maximum Lengths for Fully Integral Abutment Bridges (feet)

Pile Size	Skew ≤ 20°	
	Steel	Concrete
Piles per 5.4.2.1 with fully fixed heads	300	500

5.4.2.7 'Best Practices' for Moderate to Long Span IABs

The following 'best practices' should be considered as design features for moderate to long span integral bridges, defined as integral steel bridges longer than 200 feet and concrete bridges longer than 330 feet:

- Only straight stringers/beams should be used on long span IABs.
- The annual thermal cyclic movement of the IAB abutments results in the development of a settlement trough adjacent to each abutment as backfill soil slumps downward and toward the abutment in its winter position. To prevent the settlement of the pavement structure, approach slabs must be included in the design of moderate to long span IAB structures, to span over the void created by the settled soil.

- Provide 2 layers of polyethylene sheets, or other bond breaker, under the approach slab to minimize friction against horizontal movement. Many States recommend two layers of 4 to 6 mil thick polyethylene sheets.
- Consider pavement expansion joints to reduce distress of the approach pavement, caused by the thermal cyclic movement of the abutments and the approach slabs. Recommended cycle control joints systems that employ a combination of asphaltic plugs, asphalt impregnated fiber board, and sleeper slabs at the end of the at-grade approach slabs, or at the end of the abutment (in the case where the slab is buried).
- A bridge with a total length in excess of 300 feet will have larger movement demands. If the anticipated abutment movements are in excess of 1.0 inch, consider strong axis pile orientation to prevent a plastic hinge under weak axis bending.
- Approach slabs should also be positively attached to the abutment to prevent slabs from “walking off” corbels during annual thermal movements of the abutment.
- Pavement geotextiles can be used to add tensile strength to pavement over the abutment backwall.
- Provide adequate drainage of the abutment backfill to prevent damage due to frost action and piping of the backfill material.
- Bridge abutments with movements in excess of 1 inch may require a higher level of pile analysis to consider all applicable forces and moment demands, including thermal, skew effects and deflections of the superstructure. A dedicated check of pile capacity for combined axial loading due to dead and live load and bending stresses due to thermal superstructure movement, using LPILE[®] software may be required.
- Pre-auger to a depth of 10 feet for the top portion of piles and then fill the hole with a non-compacting backfill material, such as underdrain backfill Type C. This creates a hinge effect in the substructure and has the effect of reducing the lateral soil stiffness by increasing the depth to fixity and reducing bending moment stress in the pile.
- Long-span integral bridges receive significant support from the embankments, and therefore, they only should be built in conjunction with stable approach embankment foundation soils.
- To mitigate excessive earth pressures, limit abutment heights.

- Avoid abutments of differing height; as such a practice may promote unequal movements at the two abutments.
- Select a span arrangement and bearing types that result in approximately equal movements at each abutment.
- As a result of the soil movement, the summer lateral earth pressures tend to increase over time as the soil immediately adjacent to each abutment becomes increasingly wedged in. This phenomenon of soil wedging and long-term buildup of lateral earth pressures is referred to as “ratcheting”. To avoid potential problems, abutments should be designed for full passive pressure using Coulomb Theory.
- Limit the use of long span integral abutments to bridges with skews less than 20 degrees to minimize the magnitude and lateral eccentricity of potential longitudinal forces.
- Make wingwalls as small as practical to minimize the amount of structure and earth that have to move with the abutment.
- Configure wingwalls to minimize resistance to abutment movement.

5.4.2.8 Abutment Details

Typical integral abutment details for steel and concrete superstructures are shown in Figure 5-2 and Figure 5-3, respectively. For steel superstructures, fixed head integral abutments are preferred but pinned head abutments are allowed.

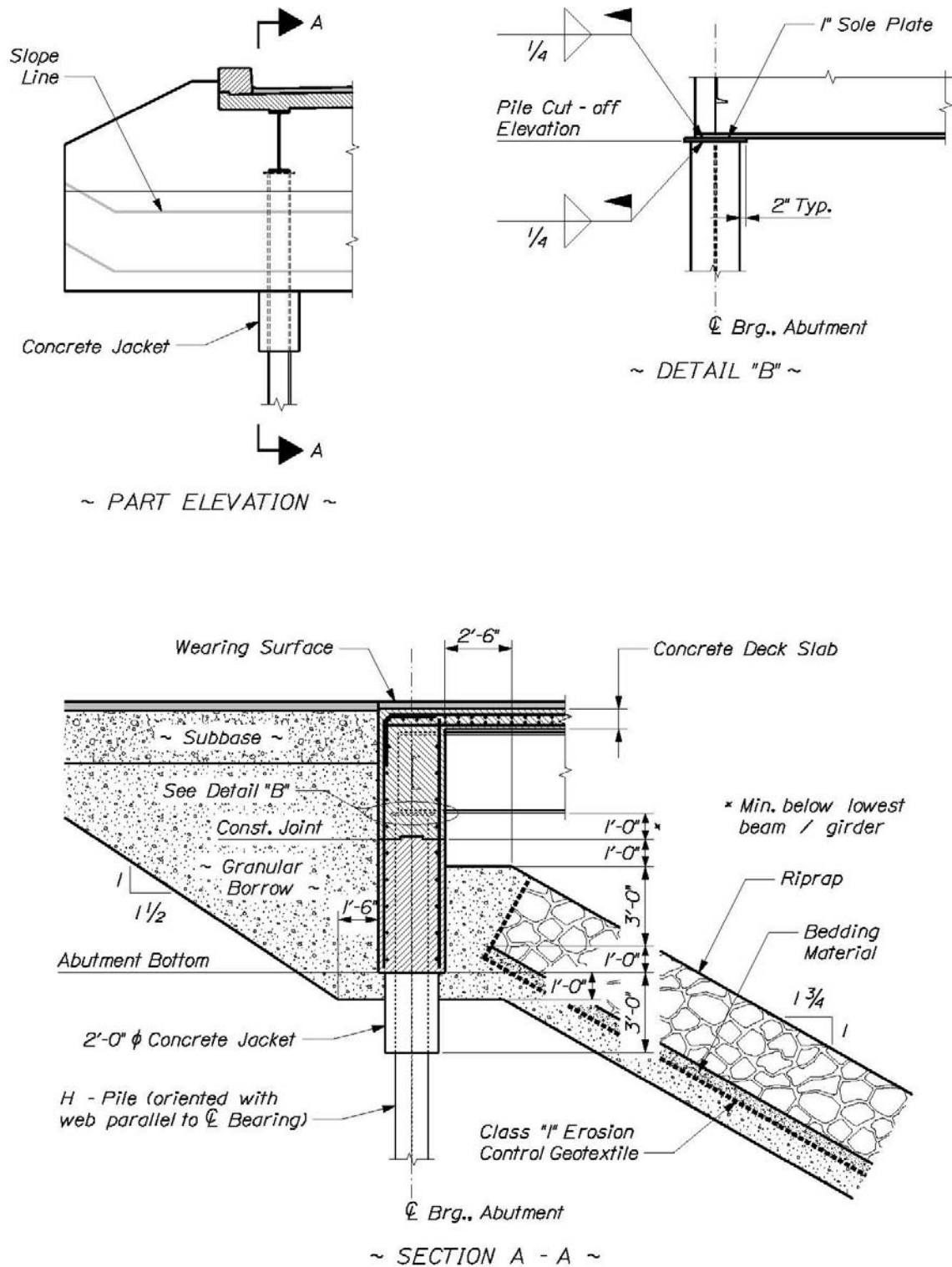
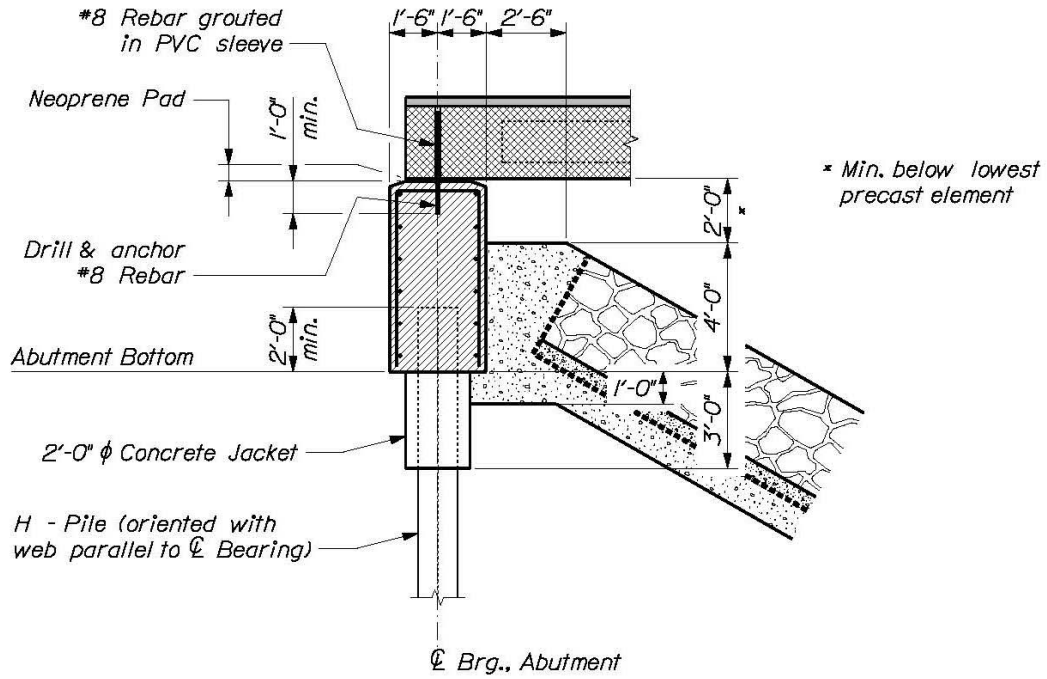
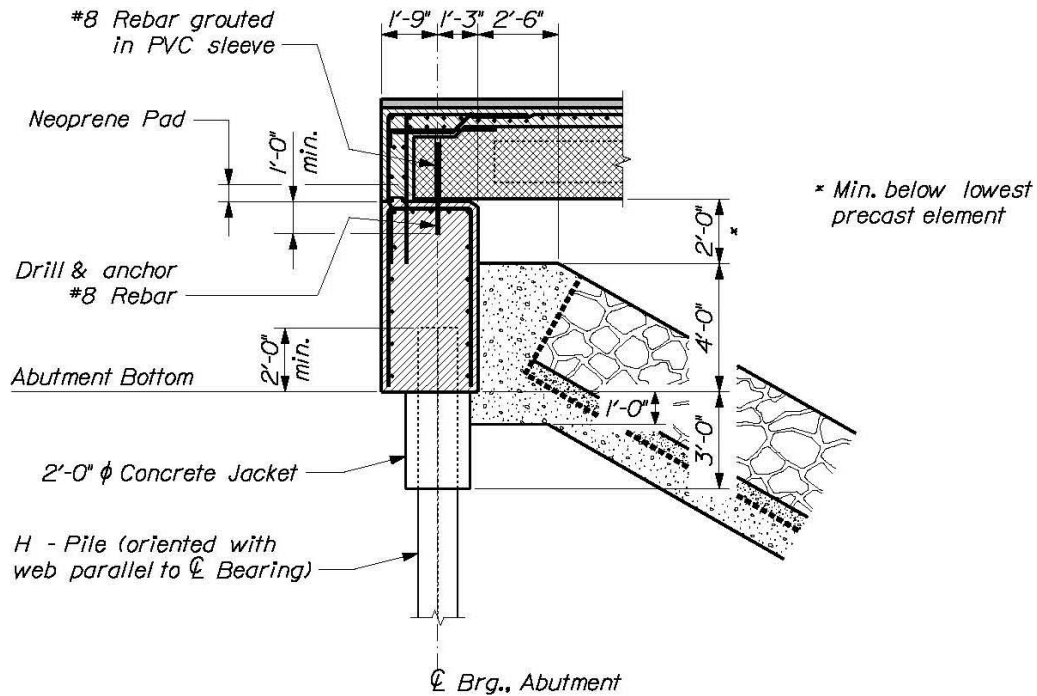


Figure 5-2 Fixed Pile Head, Full Integral Abutment Details-Steel Superstructures

CHAPTER 5 - SUBSTRUCTURES



~ INTEGRAL ABUTMENT WITH HINGE ~



~ FULL INTEGRAL ABUTMENT ~

Figure 5-3 Integral Abutment with Hinge and Full Integral Abutment Details – Precast Superstructures

5.4.2.9 Alignment

Curved bridges are allowed, provided the stringers are straight. Beams should be parallel to each other. All substructure units should be parallel to each other.

The maximum vertical grade between abutments is limited to 5%.

5.4.2.10 Superstructure Design

No special considerations should be made for integral abutment designs. Fixity at the abutments should not be considered during beam/girder design.

When selecting span ratios for multi-span bridges, consideration should be given to providing nearly equal movement at each abutment.

5.4.2.11 Abutment and Wingwall Design

Design abutment and wingwall reinforcement for the passive earth pressure (P_p) which results on the back face of the wall when the bridge expands. Refer to Section 3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient (K_p) and Table 5-4 for the passive earth pressure load factor (γ_{EH}).

Design bars for the backwall for full passive pressure due to the abutment backfill material. The backwall acts as a continuous horizontal beam supported on the piles, i.e., with spans equal to the girder spacing. Design the bars for 1) the maximum factored shear due to the factored passive earth pressure and, 2) flexure due to the moment from the factored passive soil pressure. Determine the passive pressure P_p acting on the full height of the abutment backwall (H_{abut}) from the bottom of the approach slab to the bottom of the abutment/pile cap. The passive pressure acts in a triangular pressure distribution:

$$P_p = \frac{1}{2} \cdot \gamma_{soil} \cdot H_{abut}^2 \cdot k_p$$

Design for a factored moment equal to:

$$M_{up} = \gamma_{EH} \cdot \frac{P_p \cdot l^2}{8}$$

A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, $\gamma_{EH} = 1.50$.

Design the abutment wall top and bottom horizontal bars for vertical loads, considering the wall to be a continuous beam with piles as supports.

Wingwalls should preferably be straight, cantilevered extension wings not to exceed 10 feet in length. Design wingwall reinforcement for the passive earth pressure (P_p) which results on the back face of the wall when the bridge expands, using the Coulomb passive earth pressure state and a passive earth pressure load factor (γ_{EH}) of 1.5. The use of flared wingwalls may be considered at stream crossings where the alignment of the stream would make in-line walls subject to scour. Piles should never be placed under wingwalls that are integral with the abutment stem. Generally the design is controlled by the horizontal bending in the wingwall at the fascia stringer caused by large passive pressures bending the wingwall.

Because of the high bending moments due to passive pressure in wingwalls 10 feet or longer, it may be necessary to support longer wingwalls on their own foundations, independent of the abutments. A flexible joint must be provided between the wingwalls and the backwall. U-wingwalls cantilevered from the abutment stem should only be considered to address right-of-way or wetlands encroachment. U-wingwalls should be no longer than 10 feet and tapered to reduce earth pressures. If an approach slab must extend to a U-wingwall, use a 2 inch joint with filler to separate the slab and the wall.

Developing full passive earth pressure requires that wall rotation, i.e. the ratio of lateral abutment movement to abutment height (y/H), exceeds 0.005. If the calculated rotation is significantly less than that required to develop full passive pressure, the Designer may consider using the Rankine passive earth pressure case, which assumes no wall friction. For the passive earth pressure case, wall friction acts downward against the passive wedge and increases passive pressure in the Coulomb state.

5.4.2.12 Approach Slabs

In addition to the requirements of Section 5.4.4, approach slabs should be used when integral bridge lengths exceed 80 feet for steel structures and 140 feet for concrete structures.

Provisions for movement between the approach slab and approach pavement is not necessary until bridge lengths exceed 140 feet for steel structures and 230 feet for concrete structures. Approach slabs below grade should be attached to the abutment. For at grade approach slabs, consideration should be given to the installation of an expansion device between the approach slab and the abutment. Refer to recommendations for approach slabs for moderate to long span integral bridges in Section 5.4.2.7.

5.4.2.13 Drainage

The area behind integral abutments should be backfilled with granular borrow for underwater backfill. A proper drainage system as described in Section 5.4.1.9 should be provided to eliminate hydrostatic pressure and control erosion of the underside of the abutment embankment slope protection. A drainage system is of great importance when there is potential for a perched or high groundwater condition, when the bridge is located in a sag curve, when the bridge is located in a cut section with saturated subgrade, or when there is significant pavement water runoff to side slopes. In these situations, consideration should also be given to backfilling integral abutments with gravel borrow or aggregate subbase course - gravel.

5.4.2.14 Scour

The Designer should ensure the stability of the structure for anticipated scour, as defined by LFRD 2.3.11. This may require driving the piles deeper than what is required by geotechnical criteria. The minimum pile length should be provided beyond the depth of computed scour for the check flood for scour.

5.4.2.15 Integral Abutment on Spread Footing Design

Spread footing abutments may be used only if designed and detailed as a semi-integral bridge abutment. Refer to Section 5.4.3 Semi-Integral Abutments.

5.4.3 *Semi-Integral Abutments*

A semi-integral bridge is defined as a “single span or multiple span continuous deck-type bridge with rigid non-integral abutment foundations, and with a movement system composed primarily of reinforced concrete end-diaphragms, backfill, approach slabs, and movable bearings located in horizontal joints at the superstructure/abutment interface” (TRB, 1996).

A semi-integral abutment bridge is characterized by:

- Elimination of expansion joints in the deck and roadway
- The superstructure backwall (end diaphragm) is not connected to the abutment, but moves along a bearing and horizontal joint below ground
- Thermal movement is accommodated by expansion bearings and a small vertical gap between the end diaphragm and the abutment

- The abutments are typically supported on spread footings or multiple rows of piles

Semi-integral abutments should typically be designed for active earth pressure over the rigid abutment height and a uniform pressure distribution due to the height of soil behind the superstructure. The superstructure backwall should typically be designed for full passive pressure only. In designing for active pressure, a Rankine active earth pressure coefficient, K_a , is recommended.

Semi-integral bridge design is still considered experimental, and must receive approval from the Engineer of Design during the preliminary design phase as a design exception.

Research findings have resulted in TRB design recommendations that include the following:

- Utilization of attached approach slabs and return wingwalls to lock the superstructure into the backfill
- Deliberate construction of an air space below the end diaphragms to prohibit an undesirable shift in the end reaction location

5.4.4 Approach Slabs

Approach slabs should be used on collectors and arterials, where:

- the design hour volume (DHV) is greater than 200,
- abutment heights (bottom of footing to finish grade) are greater than 20 feet, or,
- poor soil conditions are encountered and settlement is anticipated in the vicinity of the abutment.

Additional requirements for the use of approach slabs on integral abutment bridges are provided in Section 5.4.12.

Approach slab seats should be 6 inches wide and specified to have a roughened surface. Approach slab seat dowels should not be used except on integral abutments as discussed in Section 5.2.4.12. Approach slab seats should be a minimum vertical distance of 2'-9" from the roadway surface. If the backwall is very high, the Structural Designer may elect to make an optional horizontal construction joint at the approach slab seat elevation.

When a structural approach slab is specified, reduction, but not elimination, of the vehicle surcharge loads may be considered per LRFD 3.11.6.5.

5.5 Piers

5.5.1 Mass Piers

Mass piers are intermediate vertical supports, which extend from the foundation, either a spread footing or deep foundation, to a pier cap, which supports the superstructure. The primary functions of pier are:

- Support dead loads, live loads and other loads from the superstructure
- Support its own weight and other loads acting directly on the pier
- Transmit all loads to the underlying foundation

The connection between the pier and the superstructure may be pinned, fixed, or free. Mass piers are typically constructed from reinforced concrete, but may be precast. Mass piers may consist of gravity, solid wall, single-column, or multiple-column piers. Single-column and multiple-column piers are usually designed in a “hammerhead” configuration at the pier cap.

5.5.1.1 Pier Selection Criteria

Selection of the mass pier configuration is based on the following factors:

- Loading conditions
- Skew
- Slenderness, with respect to buckling
- Aesthetics
- Likelihood of debris. The use of multiple-column piers in areas where floating debris may lodge between columns should be avoided.

5.5.1.2 Load Combinations and Load Factors

Mass piers should be designed in accordance with AASHTO LRFD, including, structural design of reinforced concrete and geotechnical analysis and design, such as bearing capacity, sliding, and eccentricity (overturning). Piers should be designed and proportioned to resist all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5 and as outlined in Chapter 3.

The following load combinations should be considered as a minimum for geotechnical analysis:

- Strength I-construction. Strength Limit State I with the exception that bridge superstructure, or a portion of that, and vehicle live loads are neglected. Any anticipated staged construction loading should be investigated. Load factors for the dead load or other components shall not be less than 1.25. Live load surcharge is included to account for construction equipment live loading during structure erection, and a construction load factor of 1.5 should be assumed.
- Strength I: Normal vehicular use without wind: dead load, all applicable live load combinations, impact; braking force (for one and two lanes) centrifugal forces, static water pressure, buoyancy and stream pressure. For Strength 1, the minimum and maximum permanent load factors are selected to create the greatest force and moment effects for the mode of stability being investigated. .
- Strength III: Load combination relating to high wind velocity (100 mph) without vehicular live load: dead load, earth pressure, if applicable; buoyancy; stream flow pressure; wind; wind on live load; and longitudinal force from thermal displacements. Minimum and maximum load factors for permanent loads (γ_p) are selected to produce the extreme force or moment effect for sliding, eccentricity or axial loading analyses.
- Strength IV: Load combination relating to very high dead load to live load force effect ratios exceeding about 7.0. Minimum and maximum load factors for permanent loads (γ_p) are selected to produce the extreme force or moment effect for sliding, eccentricity or axial loading analyses.
- Strength V: Load combination relating to the bridge exposed to 55 mph wind velocity with live loads: dead load; live load plus impact; centrifugal force; earth pressure; buoyancy and stream flow pressure. . Minimum and maximum load factors for permanent loads (γ_p) should be selected to produce the extreme force or moment effect for sliding, eccentricity or axial loading analyses.
- Service I: Normal vehicular use of the bridge with a 55 mph wind load. All loads are taken at their unfactored values.

Debris loading shall be accounted for in water pressure loads by a 25% increase in the exposed surface area of the pier.

A Maine-modified Strength Limit State analysis should be performed that includes in the ice pressures of past practice, specified in Section 3.9 Ice Loads, with the appropriate resistance factors applied to the pier components. The Strength Limit State that produces the extreme force or moment should be selected.

Where piers are to be designed to resist earthquake forces, collisions by roadway or rail vehicles, vessel collision or ice, the structures should be evaluated for the following additional limit states:

- Extreme Event I – Load combination including earthquake forces, using permanent load factors, γ_p , which produce the greatest load and moment effects for the mode of stability being analyzed.
- Extreme Event II – Dead load; live load; buoyancy; static water pressure; stream flow pressure; ice pressure; vessel impact and vehicle or railway impact, using permanent load factors, γ_p , which produce the greatest load and moment effects for the mode of stability being analyzed.

For Extreme Event II apply ice force effects, and vessel, vehicle and railway collision forces one at a time since the joint probability of these events is extremely low.

The ice pressures for Extreme Event II shall be applied at Q1.1 and Q50 elevations as defined in Section 3.9 Ice Loads with the design ice thickness increased by 1 foot and a load factor of 1.0.

The critical load conditions for the evaluation of foundation bearing capacity, overturning (for pile foundations assess uplift loading of piles), eccentricity, and sliding (lateral loading for deep foundations) are those combinations of minimum or maximum loads and moments which produces the maximum force or moment effect.

With regards to vehicular live load (LL and IM) lane placement is important and multiple presence factors (MPF) are applicable. Impact forces should only be applied to truck or tandem loads:

- IM = 0.33 for cap and stem
- IM = 0 for buried footings

In consideration of the potential deflections due to bending of a pier about its weak (transverse) axis may result in magnification of the longitudinal moments on the pier, the Designer should compute longitudinal moment magnification factors for each load combination and Strength Limit State

based on the factored loads and pier stiffness. The Moment Magnification Factors are provided in LRFD 4.5.3.2.2.

5.5.1.3 General

The designer should estimate the load combinations which could be imposed on the pier and estimate the nominal resistance of the structural component or ground. Pier components shall satisfy the following equation for each limit state:

$$\sum \eta_i \gamma_i Q_i \leq \Phi R_n = R_f$$

where:

- η_i = Factors to account for ductility, redundancy and operational importance
- γ_i = Load factor (dim)
- Q_i = Force effect or stress (kip)
- Φ = Resistance factor (dim)
- R_n = Nominal resistance (kip)
- R_f = Factored resistance (kip)

5.5.1.4 Strength Limit State Evaluations

The above equation should be used to evaluate piers and pier foundations at the strength limit states for:

- Bearing resistance failure
- Lateral sliding
- Excessive loss of base contact (eccentricity)
- Pile group failure
- Structural failure

The factored resistance, R_f , calculated for each mode of failure, is to be calculated using the appropriate resistance factors for bearing resistance, sliding, eccentricity, axial pile resistance and structural resistance.

The Designer should consider the consequences of changes in the pier foundation conditions from scour due to the design flood event using appropriate strength limit state resistance factors. Debris loading during flood events should be accounted for in water pressure loads by assuming a 25% increase in the exposed surface area of the pier.

The investigation of piers at the strength limit states for structural failure should be in accordance with LRFD Article 5.7 and carry all flexure and axial loads anticipated. Appropriate consideration should be given to the effects of slenderness on both aesthetics and load-carrying capacity.

For piers founded on piles, the shear on the critical section should be investigated at the strength limit state in accordance with AASHTO LRFD Section 5.13.3.6.

5.5.1.5 Service Limit State Evaluations

Piers should be investigated at the service limit state for:

- Settlement
- Lateral displacement
- Overall slope stability
- Foundation stability, settlement and horizontal movement at the design flood for scour

A resistance factor, ϕ , of 1.0 is used to assess pier design at the service limit state. The overall global stability of the foundation should be investigated at the Service Load Combination with a resistance factor, ϕ , of 0.65.

Tolerable vertical and lateral displacement criteria for piers shall be developed based on the function and type of pier, anticipated service life, and consequences of unacceptable movements of the pier and effect on the superstructure and bearings.

5.5.1.6 Extreme Event Limit State Evaluations

Extreme event limit state design checks for piers should include:

- Bearing resistance
- Eccentricity
- Sliding
- Overall stability
- Pile group failure
- Structural failure

A resistance factor, ϕ , of 1.0 is used in the load and resistance equation in Section 5.4.1.3 to assess pier design at the extreme limit state.

Resistance factors for extreme event limit states shall be taken as 1.0.

For the extreme event limit state, the Designer should consider scour due to the check flood event and should determine that there is adequate foundation resistance to support all applicable unfactored loads with a resistance factor of 1.0 or less. Debris loading during flood events should be accounted for in water pressure loads by a 25% increase in the exposed surface area of the pier.

5.5.1.7 Structural Design

The structural design of piers shall be in accordance with the provisions of LRFD Sections 5, 6, 7, and 8, as appropriate.

The investigation of piers at the strength limit states for structural failure should be in accordance with LRFD 5.7 and carry all flexure and axial loads anticipated. Appropriate consideration should be given to the effects of slenderness on both aesthetics and load-carrying capacity.

For piers founded on piles, the shear on the critical section should be investigated at the strength limit state in accordance with AASHTO LRFD 5.13.3.6.

5.5.1.8 Structural Design of Columns

The primary checks for a pier shaft or column structural design consist of:

- Determine maximum moments and shears in the shaft/column
- Check limits for reinforcement (LRFD 5.7.4.2)
- Calculate the factored axial resistance (LRFD 5.7.4.4)
- Check slenderness provisions for compression members (5.7.4.3)
- Calculate the moment magnification factors (LRFD 4.5.3.2.2b)
Develop shaft or column interaction curve
- Check biaxial flexure provisions for non-circular members (LRFD 5.7.4.5)
- Determine transverse reinforcement for compression members (LRFD 5.10.6 or 5.7.4.6)

5.5.1.9 Geotechnical Design of Pier Foundations

A. Spread Footings

In using spread footings for foundation support for mass piers, either on soil or bedrock, the design should be in accordance with the AASHTO LRFD and Section 5.3 Spread Footings.

B. Deep Foundations

Deep foundations for mass piers may consist of piles or drilled shafts. Piles may consist of H- or pipe pile steel sections, or precast concrete. In founding a mass pier on a deep foundation, design should be in accordance with the AASHTO LRFD, and BDG Sections 5.7 Piles and 5.8 Drilled Shafts. In designing deep foundation elements for a mass pier with an unsupported length, a complete analysis of the foundation should be performed using actual loading and soil conditions.

For strength and extreme limit state analyses, maximum factored axial pile loads and stresses should be computed using 3-D pile group analysis software, such as FB-Multipier[®].

For service limit state design of deep foundation, a complete deflection analysis of a driven pile foundations should be performed using LPILE[®] or FB-Multipier[®] software.

C. Scour

For scour protection of mass piers in water channels, the following treatments should be considered: 1) the use of a deep seal placed minimum of 2 feet below the scour depth determined for the check flood for scour, or 2) designing the deep foundation elements for an unsupported length. The unsupported pile length should be the vertical distance from the bottom of the seal to the scour depth determined for the check flood event. Piles should achieve axial capacity and lateral capacity/fixity below the scour depth determined for the design flood event.

5.5.1.10 Pier Protection

A. Collision Forces

Where the possibility of collision exists from vehicular, railroad, or water traffic, an appropriate risk analysis should be made to determine the degree of impact resistance to be provided and/or the appropriate protection system.

Unless the department determines that site conditions indicate otherwise, or unless protected by collision walls as specified in paragraph B. below, piers located within a distance of 30 feet to the edge of roadway or within a distance of 50 feet to the centerline of a railway track shall be designed for an equivalent static force of 400 kips, which is assumed to act in any direction in a horizontal plane, normal to the wall, at a distance of 4 feet above the ground.

B. Collision Walls

The provisions of the paragraph above need not be considered for piers or abutments protected by an:

- An embankment
- A structurally independent crashworthy ground mounted 54 inch high barrier, located within 10 feet of the pier, or
- A 42 inch high barrier located at more than 10 feet from the pier

C. Vessel Collision

All bridge components in navigable waterway crossings where vessel collision is anticipated shall be designed for a specified degree of vessel impact damage in accordance with LRFD 3.14, or adequately protected by dolphins, fender systems or other sacrificial devices.

D. Scour

The majority of bridge failures in the United States are the result of scour. The added cost of making a bridge less vulnerable to scour is small in comparison to the total cost of a bridge failure.

LRFD 3.7.5 requires that scour at bridge piers be investigated for two conditions:

- For the design flood for scour, the streambed material above the total scour line shall be assumed to have been removed. The design flood storm event shall be the more severe of the 100-year event or from an overtopping flood of lesser recurrence interval. The strength and service limit states apply.
- For the check flood for scour, the stability of pier foundations shall be investigated for scour conditions resulting from a designated flood event, not to exceed the 500-year event or from an overtopping flood of lesser recurrence. The extreme event limit state shall apply. Reserve capacity beyond that required for stability under this condition is not necessary. The exception is

spread footings on soil or erodible rock, which shall be located to that the bottom of the footing is below the scour depth determined for the check flood for scour.

Refer to Section 2.3.11 Scour for additional guidance.

E. Facing

Where appropriate, the nose of the pier should be designed to effectively break up or deflect floating ice or debris. Pier life can be extended by facing the nose with steel plate/angle or by facing the pier with granite.

5.5.2 *Pile Bent Piers*

Pile bent piers are significantly less expensive than mass concrete piers and provide environmental advantages by eliminating cofferdam work and its associated impacts. Pile bents should be used wherever possible based upon the criteria below.

5.5.2.1 Pile Bent Use Criteria

Pile bent piers should not be used in the following locations:

- In rivers known for severe ice conditions - Allagash, Androscoggin, Aroostook, Kennebec, Penobscot, St. Croix, and St. John
- Other locations with severe ice conditions
- In shipping channels
- Where the pier is not aligned with the design flow

Pile bent piers should be considered for structures in the following locations:

- In tidal rivers
- In environmentally sensitive areas
- For grade-separated structures
- Within the headwater or tailwater of dams or lakes, except when ice has been known to form predominantly on one side of the pier with an open channel in the adjacent span, resulting in static ice forces on all piles.

The following issues affect the design of pile bent piers and must also be considered when evaluating the appropriateness of this system.

- Pile length - The pile length is a function of the depth to bedrock, loading conditions, the type of overburden material, the depth of scour, degree of pile fixity and restraint, and the pile bracing.
- Pile loads - The following issues affect pile loads:
 1. Application location and magnitude of ice load
 2. Skew - Longitudinal superstructure forces are transmitted into the longitudinal pier axis and increase with greater skew angles.
 3. Bridge width - Pier cap shrinkage forces increase with increasing bridge width.
 4. Span length - Dead and live load axial forces are dependent upon span length.
 5. Seismic forces.

An additional issue to be considered when evaluating the appropriateness of pipe pile pier bents is corrosion. Special consideration should be given to corrosion and abrasion of steel pile bent piers to ensure a minimum 75 year structure life is achieved. This is of particular concern in locations where there is insufficient water to install cathodic protection in accordance with Section 5.5.2.6, and in locations where debris or sediment loads may abrade pile protective coatings. In these locations the design should consider additional protection such as encased H-piles with sacrificial steel pipe pile or sacrificial fiber reinforced polymer (FRP) composite pipe pile casings.

5.5.2.2 Loads and Load Combinations

Pile bent piers should be designed in accordance with AASHTO LRFD. Structural analysis and design of reinforced concrete should include pile bent cap flexure and shear checks, pile structural resistance and buckling and lateral stability of piles. Geotechnical design checks should include strength limit state checks and service limit state checks such as global stability, horizontal bent displacement and pile settlement.

Where applicable, consideration should be given to other loading conditions, including seismic forces resulting from earthquake loading and debris lodged against pier, as outlined in 5.5.1.2 Load Combinations.

Pile bent piers should be designed and proportioned to resist all applicable load combinations specified for mass piers in 5.5.1.2 Load Combinations and Load Factors, and as outlined in Chapter 3 Loads and LRFD Articles 3.4.1, 11.5 and 11.7.

A. Live Loads

Vehicular live loads must be located within the design lanes on the superstructure such that maximum forces occur in the pile cap and piles.

Impact should be applied to pier caps and that the portion of the piles that are acting as columns, defined as the vertical distance from the pile cap to the point of fixity below grade. Impact should be applied at or above Q1.1.

B. Ice Loads

For the Extreme Event II load combination, unfactored ice loads should be placed at the Q50 stage elevation and checked at a lower elevation that will cause maximum moment in the nose pile, provided the elevation is at or above Q1.1. The ice thickness of past practice should be increased by 1.0 foot.

Transverse ice loads should be applied to only the nose pile when ice is directly applied to the nose pile, or be uniformly distributed over the cap when ice is applied to the cap.

A modified Strength Limit State analysis should also be performed with factored ice loads following the criteria specified in 3.9 Ice Loads, with appropriate strength limit state resistance factors for the pier component being analyzed.

C. Water Loads

Stream pressure should be reduced when the ice elevation is lowered to check maximum moment in the nose pile.

Stream pressure should be applied to each pile in the bent, using an appropriate stream flow velocity.

D. Wind Loads

Longitudinal components of wind on superstructure and wind on live load should be distributed to the abutments when structure fixity is at the abutments.

E. Seismic Loads

Seismic loads transverse to the bridge should be shared between all substructure units based upon their stiffness.

Longitudinal seismic loads should be distributed to the abutments where there is at least one fixed abutment with no forces applied to the pier.

F. Shrinkage and Temperature Forces

Shrinkage and temperature forces affect pile bents in two ways:

- Pile cap shrinkage and temperature actions are applied to the longitudinal axis of the pier.
- Thermal forces induced by the superstructure are applied along both the transverse and longitudinal pier axes, with the magnitude dependent upon the skew angle.

Two-span integral abutment bridges will have no associated thermal forces applied, as the forces are assumed to be balanced at the pier. The Structural Designer may want to include thermal forces for two-span integral abutment bridges on steep grades, assuming that the bridge will expand and contract downhill.

For non-integral abutment bridges, thermal forces induced by the superstructure bending the pile bents must be considered in the design of the fixed abutment.

G. Braking Forces

If the structure is fixed at an abutment, the longitudinal braking forces will have no effect on the pier, as the forces are assumed to be distributed to the abutments.

H. Friction Forces

Friction forces resulting from all longitudinal superstructure forces should be applied to pile bents with expansion bearings.

I. Collision Loads

Where the possibility of collision exists from vehicular, railroad, or water traffic, an appropriate risk analysis should be made to determine the degree of impact resistance to be provided and/or the appropriate protection system.

5.5.2.3 Pile Cap Design

Pile bent cap design should consider the following design features:

- Piles should be embedded at least 12 inches
- Pile clearance with 6 inches of concrete cover
- Tolerance on pile installation misalignments $>$ or $=$ 2 inches
- Consider concrete pile anchorage
- Pile spacing should be at least 30 inches or 2.5 times the pile diameter

5.5.2.4 Pile Type Selection Criteria

Concrete filled pipe piles, precast concrete piles, combination H-piles encased with pipe piles filled with concrete, and drilled shafts may be considered for pile bent piers under the following conditions:

- A. Shallow overburden depth (embedment less than or equal to the fixity depth)
 - Footing-encased pipe or precast concrete piles
 - Rock-socketed pipe piles
 - Rock-socketed H-piles, with pipe pile encasement to top of bedrock
 - Rock-anchored/doweled pipe piles (Note: AASHTO LRFD is absent of discussion on the use of rock-anchor pipe piles. The use of rock-anchored pipe piles should be considered only when the preceding alternatives are found not feasible. Rock anchors or dowels should have double corrosion protection.)
 - Rock-socketed drilled shafts
- B. Intermediate overburden depth (embedment greater than depth to fixity and less than 3 times fixity depth)
 - Pipe piles filled with concrete and a reinforcing cage (The reinforcing cage may be eliminated with the approval of the Engineer of Design.)
 - Precast concrete piles

- Drilled shafts
- C. Deep overburden depth (embedment greater than 3 times fixity depth)
 - Pipe piles filled with concrete and a reinforcing cage (The reinforcing cage may be removed with the approval of the Engineer of Design.)
 - H-piles with pipe pile encasement to pile fixity depth
 - Precast concrete piles
 - Drilled shafts

The choice of steel versus concrete piling in intermediate and deep applications should be determined by a cost analysis. Issues include the relative costs of H-piles to precast concrete piles or pipe piles, encasement and the relationship between the exposed length (including the scour depth), the depth to fixity, and the total depth to bearing.

D. Pier Bent Pile Alternatives

Because of ongoing corrosion and durability issues with steel pipe piles, Geotechnical Engineers and Designers should routinely examine the feasibility and practicality of other pier bent pile-types, namely:

- precast concrete piles
- drilled shaft pier bents
- encased H-piles with a sacrificial steel pipe pile or a sacrificial fiber reinforced polymer (FRP) composite pipe pile casing

5.5.2.5 Pile Protection

A. Encased H-Piles

Steel H-piles should not be used for piers without full encasement protection. The encasement usually is a steel pipe pile filled with concrete. H-piles should be protected by a minimum of 3 inch clear encasement from the pier cap to a minimum of 10 feet below streambed or 2 feet below the total scour depth. Due to the significant additional load section provided by the composite steel and concrete section, the pipe pile should be used for strength. If the pipe pile is used for strength, it should extend to the point of fixity below streambed.

The pipe pile should be protected and designed as detailed in Paragraph B. Pipe Piles, below.

B. Pipe Piles

Pipe piles bents in fresh water environments should be hot-dipped galvanized with UV-resistant epoxy top coat. Pipe pile bents in brackish or salt water should be coated with fusion bonded epoxy paint with a coat thickness of 18-20 mils. This is an increase in the previous standard of 12 mils.

Fusion-bonded epoxy coatings or galvanized surfaces should be applied to a minimum of 10 feet below streambed or 2 feet below the total scour depth.

Cathodic protection (aluminum anodes) should always be used in addition to the protective coatings in salt and fresh water environments as long as there is sufficient water to submerge the anodes at low water.

Refer to 5.5.2.6 Pipe Pile Coatings and Cathodic Protection for detailed recommendations.

C. Precast/Prestressed Concrete Piles

Concrete cover for rebar should be a minimum of 2 inches for fresh water locations and 3 inches for salt water locations.

5.5.2.6 Pipe Pile Coatings and Cathodic Protection

A. Standardized Anodes

Pipe pile pier bents and cargo/ferry piers should specify a standard anode ingot length, composition (aluminum alloy plus minor constituents) and weight.

The standard should be a 34-lb, aluminum alloy anode, approximately 3 feet long. Larger, heavier anodes are not easy to handle and should be avoided unless the bent has a lot of uncoated steel or the project is a significant sheet pile structure where there is a greater chance for exposed steel. On large piles with long exposed lengths (deep water), consideration should be given to installing more than one anode rather than using a heavier anode.

There are a lot of variables in the rate of corrosion between sites, and it may happen that the standard anode may not be suitable for all sites. Larger anodes may be necessary for more aggressive environments (brackish and saltwater). Specifying a heavier anode may be required.

B. Anode Location

The top of the 34-lb, 3-foot long anode should be 3 feet below Low Water, so that it is always submerged. This implies the water channel needs to have at least 6 feet of water at Low Water.

Anodes should be installed on the more protected side of the pile: on the underside on battered piles, on the downstream side on plumb piles in rivers, and on the more protected side (if there is one) on plumb piles in tidal crossings. If possible, show the location of the anodes on the Plan drawings, so there is no debate in the field about what constitutes the 'more protected side'.

C. Shallow Water Situations

If there is not enough depth of water to submerge the anodes at all times, the anodes are not as effective in protecting the pile segment above the waterline.

Where the water is shallow and there is no submerged portion or a limited submerged portion of pile for anodes, Designers should consider:

- specify a non-standard, shorter ingot if that permits installation on a pile in shallow water
- fusion-bonded epoxy treatment over hot-dipped galvanized piles
- encasing H-piles with a sacrificial steel pipe pile or a sacrificial FRP composite pipe pile casing

D. Anode Attachment

Plans should specify a 2-inch clearance between the anode and the pile. This allows Bridge Inspectors to get a clear view of the anode, and the pile surface is more "inspectable" and the anodes easier to replace. Attachment hardware consisting of a 3-inch long, $\frac{3}{4}$ -inch diameter threaded stud, with double nuts, is recommended. The studs should be installed in a manner that ensures the best steel to steel connection and the best electrical connection. The weld area shall be ground to bare metal for this purpose. Only after the stud and anode are attached, shall the weld at the base on the stud be covered with curable polyamide epoxy coating.

E. Brackish and Saltwater Environments

Steel pile bents in brackish or salt water should not be hot-dipped galvanized with UV-resistant epoxy top coat. These pile bents should be coated with fusion bonded epoxy coating with a thickness of 18-20 mil.

Cathodic protection aluminum anodes should always be installed on pile pier bents in salt water when there is enough water.

F. Freshwater Environments

Steel pile bents in freshwater should be hot-dip galvanized with a UV-resistant topcoat system. The UV-resistant topcoat tends to fade where the upper part of the pile gets direct sunlight and reflected light from the water surface. Considerations should be given to topcoating with fluorocarbon paint, which is more UV resistant.

Cathodic protection aluminum anodes should be installed on pile pier bents in fresh water, with the exception of river crossings with very shallow water.

G. Coating Repairs

Pile coating “touch-up” per the manufacturer’s recommendations is considered the best practice for dealing with damaged pile sections. The “touch-up” material on some jobs (in particular, Alna-Newcastle) seems to be performing well. The Bridge Program should determine the best “touch-up” method and specify it – not just specify *“touch-up per Manufacturer’s recommendations”*.

H. Pipe Pile Material

Steel pipe piles should be ASTM 252 and have straight butt-welded seams or be seamless. Spiral seams are not recommended because the magnitude of welded surfaces which are vulnerable to thin coatings, ice abrasion, and bumping during construction – all of which lead to damage in the coating. Welds should be ground down and blended smoothly with the pile material. The number of field and mill splices should be limited.

I. Damage during Construction

Specifications should include requirements for Contractors to line driving templates with fire hoses, carpets, etc., to prevent the scraping off of the coatings during pile driving. Contractors should be required to repair or replace any protective mats that fall off during driving, prior to commencing driving any more pile.

5.5.2.7 Additional Pile Bent Pier Design Criteria

Pile bents should consist of a concrete pile cap supported by a single row of piles, multiple rows of piles, or a braced group of piles.

A. Pile Design

Pile design should investigate resistance to axial loads, combined axial and bending, and buckling failure of the exposed pile lengths. Guidance for computing the unsupported pile length is provided in Section B, below. Stability of the pile bent pier under combined axial and lateral loads should be investigated with a dedicated soil-structure interaction analysis, using FB-Pier software.

B. Pile Length

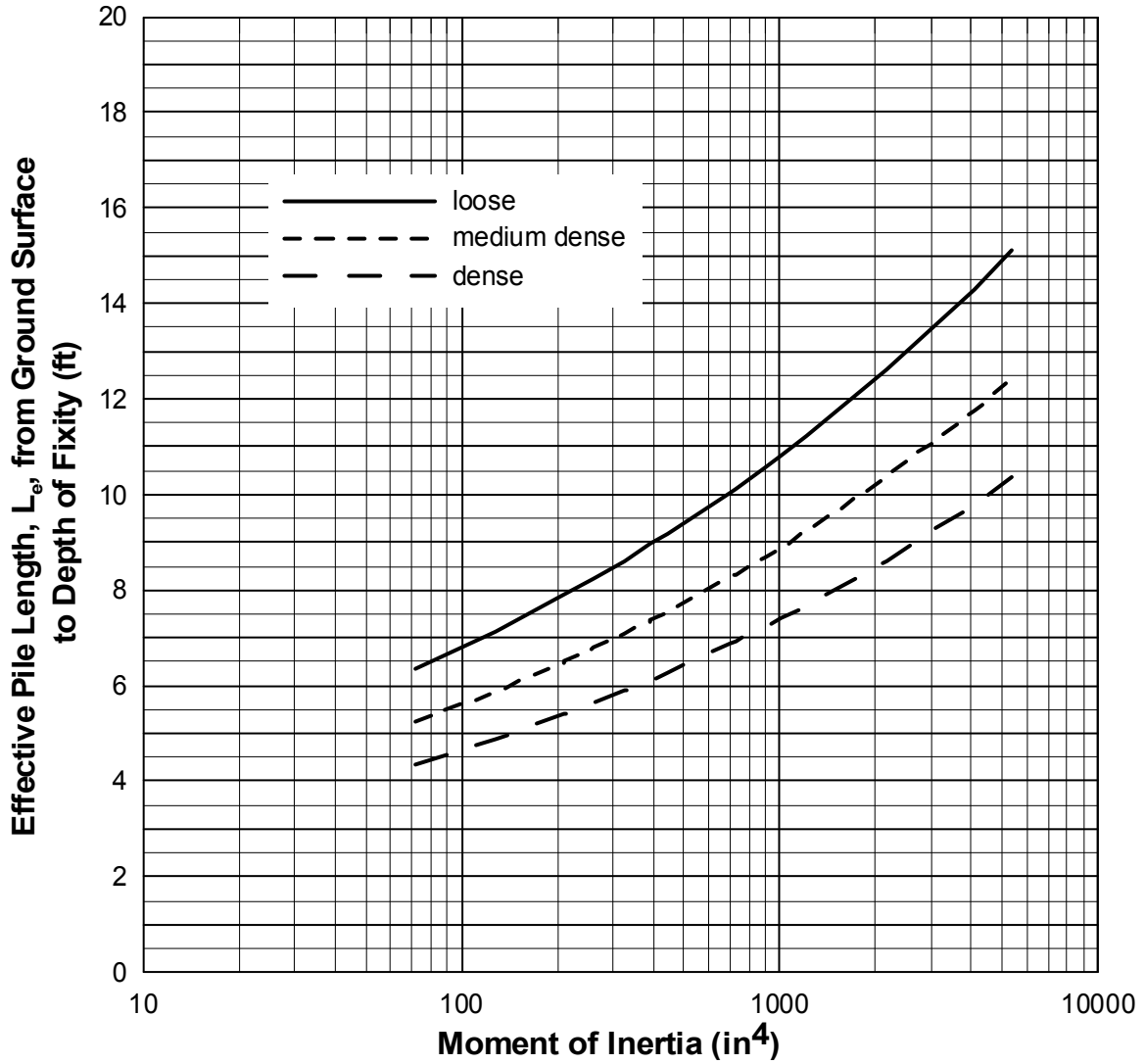
The unsupported length, L_{us} , is defined by the following:

$$L_{us} = K \cdot (L_u + L_e)$$

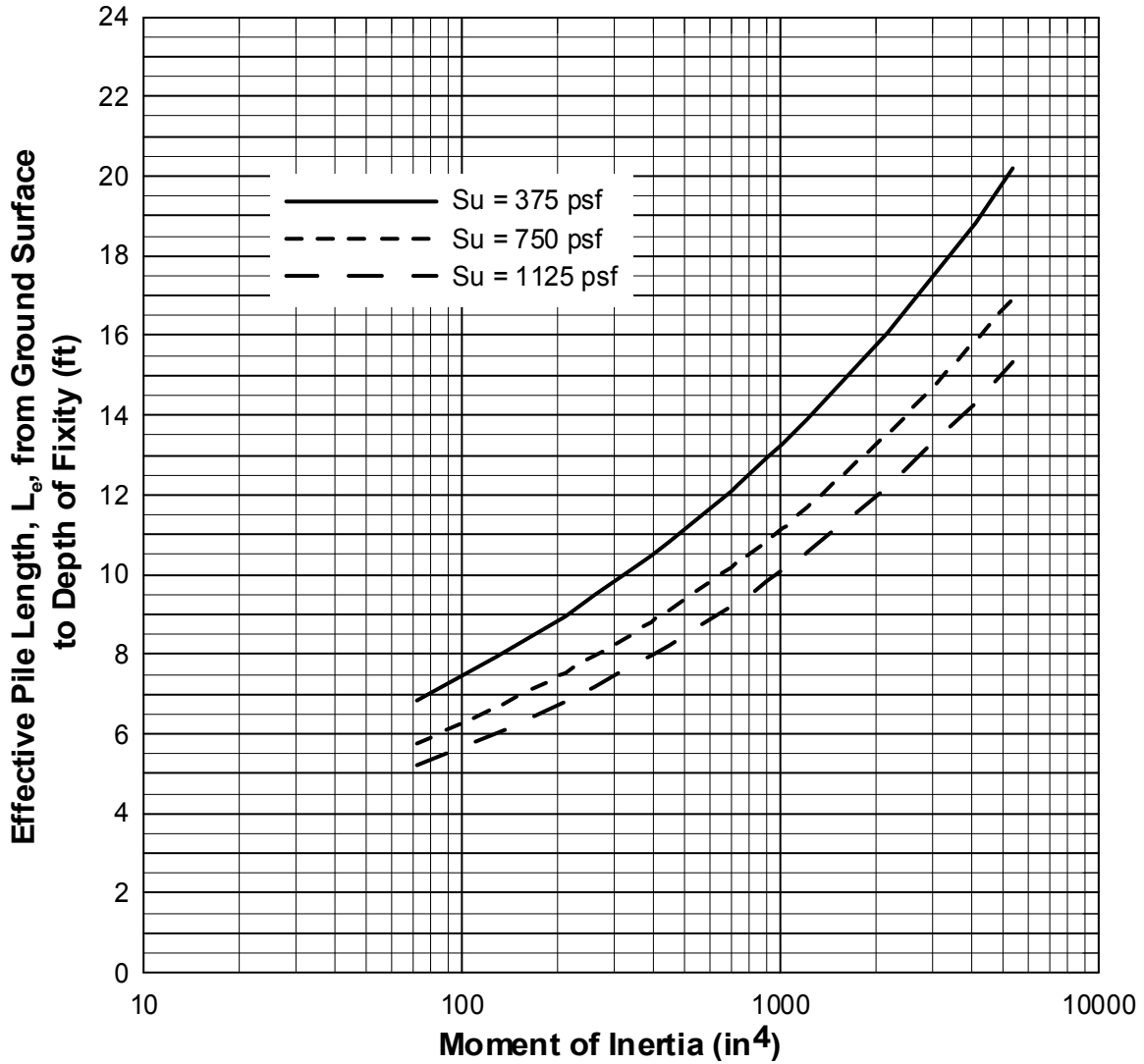
where:

- K = Effective Length Factor. Refer to LRFD Article 4.6.2.5 and Table C4.6.2.5-1.
- L_u = Exposed pile length above ground.
- L_e = Effective pile length from ground surface to the point of assumed fixity below ground, including scour effects. Refer to Figure 5-4 and Figure 5-5.

The depth to fixity shown in Figure 5-4 and Figure 5-5 was determined using the Davisson and Robinson procedure provided in LRFD Article 10.7.3.13.4 and assumes no lateral loading on the pile. Where piles used for pile bent piers are subjected to lateral loading or where the embedment length is less than $3L_e$, a detailed analysis by the Designer using actual loading and soil conditions is required.



**Figure 5-4 Effective Pile Length for Piles in Sand
From Ground Surface to Depth of Fixity
Axially Loaded**



**Figure 5-5 Effective Pile Length for Piles in Clay
From Ground Surface to Depth of Fixity
Axially Loaded**

C. Nose Pile Batter

Where possible, the nose pile should be battered a minimum of 15° to take advantage of the allowance for ice load reduction due to nose inclination (refer to LRFD Article 3.9.2.2). When ice is applied to the pier cap or within 5 feet of the pier cap, no reduction should be taken.

D. Design Section

Encased H-piles and concrete-filled pipe piles should be designed assuming contribution from the concrete and a portion of the steel pipe pile shell, allowing for a minimum of 0.15 inch of sacrificial shell

corrosion, based on a design corrosion rate of 0.05 mm per year. The pipe pile shell must have a minimum thickness of 1/2" to allow for proper driving of the pile and to resist corrosion.

MaineDOT Section 711.01 specifies ASTM 252 for Welded and Seamless Steel Pipe Piles. Designers should consider that ASTM 252 permits under-fabrication of the wall thicknesses up to 12.5% of the specified nominal wall thickness. Example: If the design calls for 5/8-inch wall, the design section should be reduced by a minimum 1/8-inch for sacrificial steel shell corrosion and an additional 1/16-inch to account for permissible fabrication variation.

5.6 Retaining Walls

5.6.1 General

Retaining walls typically used by the Bridge Program are gravity walls, cantilever-type walls, mechanically stabilized earth (MSE) walls, prefabricated proprietary walls and soil nail walls, each of which is discussed in detail in the following sections. The selection of the appropriate retaining wall should be based on an assessment of the magnitude and direction of loading, depth to suitable foundation support, potential for earthquake loading, presence of deleterious factors, proximity of physical constraints, wall site cross-section geometry, tolerable and differential settlements, facing appearance, and ease and cost of construction. A feasibility study should address which wall is most suited to the site and is simplest to construct. The study should address the approximate scope of the design for the most feasible walls, and provide cost comparison between alternatives.

5.6.1.1 Retaining Wall Type Selection

Due to construction techniques and base width requirements, some wall types are best suited for cut sections whereas others are best suited for fill situations. The key considerations in deciding which wall is feasible are the amount of excavation or shoring required and the overall wall height. The site geometric constraints must be well-defined to determine these elements.

A. Walls in Cut Sections

Anchored walls and soil nail walls, which have soil reinforcements drilled into the in-situ soil/bedrock, and cantilever sheet pile walls, are generally used in cut situations. These walls are typically constructed from the top down.

B. Walls in Fill Sections

MSE walls are constructed by placing soil reinforcement between the layers of fill from the bottom up and are therefore best suited to fill situations. Additionally, the base width of MSE walls is typically on the order of 70% of the wall height, which would require considerable excavation in a cut section, making the use of this wall uneconomical.

C. Walls in Cut or Fill Sections

Gravity, cantilever-type, and prefabricated proprietary walls are freestanding structural systems built from the bottom up that do not rely on soil reinforcement techniques to provide stability. These types of walls have a narrower base width than MSE structures (on the order of 50% of the wall height) making this type of wall feasible in fill situations as well as many cut situations.

5.6.1.2 Service Life

Retaining walls should be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents, and other potentially deleterious environmental factors on each of the material components comprising the wall. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining walls for temporary applications are typically designed for a service life of 36 months or less. Greater level of safety and/or longer service life (i.e., 100 years) may be appropriate for walls that support bridge abutments, for which the consequences of poor performance or failure would be severe.

The quality of in-service performance is an important consideration in the design of permanent retaining walls. Permanent walls should be designed to retain an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life.

5.6.1.3 Design Loads

Retaining walls should be designed in accordance with AASHTO LRFD Structural analyses, the design of reinforced concrete and geotechnical analyses of retaining walls will be computed using LRFD procedures using factored loads and factored resistances. The geotechnical design of conventional retaining walls typically follows the LRFD approach for the design of abutments on spread footings, presented in 5.3 Spread Footings and 5.4 Abutments. Where a wall is supported with piles or dilled shafts, the design will follow LRFD and 5.4.1.12 Abutments Supported on Pile Foundations and 5.7 Piles, as appropriate. Loads should be determined in accordance with AASHTO LRFD and as outlined in Chapter 3 and

5.4.1.2 Load Combinations and Load Factors. The following load conditions should be considered when applicable:

- Lateral earth pressure
- Weight of soil above the footing or within the wall system
- Self-weight of the wall
- Lateral loads due to live load impact on the parapets
- Surcharge loads, due to live load
- Surcharge load caused by earth, point, line or strip loads on the upper surface
- Railroad loading
- Hydrostatic pressure (if no drainage is provided)

Earth pressure due to compaction should be considered when static or dynamic compaction is used within a distance of one-half of the wall height. These loads will only apply to during construction phase; therefore a load factor of 1.0 is appropriate.

Walls that can tolerate little or no movement, or are restrained, should be designed for at-rest (K_o) earth pressure with a maximum load factor for at-rest earth pressure, γ_{EH} , of 1.35.

5.6.1.4 Limit States

Retaining walls should be designed to resist all applicable load combinations specified in LRFD 3.4.1 and 11.5.5.

Strength limit state checks of walls should assess external failure mechanisms:

- Sliding
- Eccentricity
- Bearing Resistance
- Structural Capacity

Service limit state check should assess overall stability, wall settlement and lateral displacement.

Walls should be evaluated for each of the applicable limit states:

- Strength I-construction. Strength Limit State I which models the basic load combination related to construction loads. Load factors for the dead load of other components shall not be less than 1.25. Live load surcharge is included to account for construction equipment live loading; a construction load factor of not less than 1.5 should be assumed.
- Strength I-a: Strength Limit State I, which models the basic load combination related to normal vehicle live load surcharge, dead load plus earth pressure, finished grade, including any point or strip loads on the wall backfill. Minimum vertical permanent load factors and maximum horizontal load factors are selected to produce extreme force effects for wall sliding and eccentricity, and structural design of the wall stem.
- Strength 1-b: Strength Limit State 1 as described above, except maximum vertical permanent load factors, including earth loads, are selected to produce an extreme force effect for bearing capacity analyses.
- Service I: Service Limit State I – Load combination relating to normal operational use of the wall with all loads taken at their unfactored values.

Wall foundations subject to scour should be designed at the strength and service limit states so that there is adequate foundation resistance, in conjunction with the depth of scour from the design flood, using appropriate strength and service limit state resistance factors.

The consequences of changes in wall foundation conditions due to scour from the check flood for scour should be assessed at the extreme event limit state with resistance factors of 1.0.

Where retaining walls are to be designed to resist earthquake forces, collisions by roadway or railway vehicles, or vessel collision, the structures should be evaluated for the following additional limit states:

- Extreme Event I – Load combination including earthquake forces
- Extreme Event II – Load combination relating to collision by vehicles, railways or vessels.

Each load for each limit state above is modified by the prescribed load factor, γ . Certain permanent loads, including earth loads, should be factored using the load factors γ_p . Load factors should be selected to produce the total extreme factored force effect. Applicable load factors,

load combinations and the analyses for which they will govern, are provided in Table 5-6.

Table 5-6 Typical Load Groups and Load Factors

Load Group	γ_{DC}	γ_{EV}	γ_{LSV}	γ_{LSH}	γ_{EH} (active & passive)	γ_{EH} (at-rest)	γ_{ES}	Typical Geotechnical Analysis Governed
Strength I-a	0.90	1.0	1.75	1.75	1.5	1.35	1.5	<ul style="list-style-type: none"> - Sliding - Eccentricity (overturning) - Structural design of wall stem - Anchor pullout
Strength I-b	1.25	1.35	1.75	1.75	1.5	1.35	1.5	<ul style="list-style-type: none"> - Bearing Capacity - Structural design of the wall footing
Service I	1.0	1.0	1.0	1.0	1.0	1.0	1.0	<ul style="list-style-type: none"> - Settlement - Lateral displacement - Global stability

5.6.1.5 Strength Limit State

A. Bearing resistance

The check for bearing resistance for wall spread footings on soil or rock is identical to the requirements for abutments described in 5.3.5 Bearing Resistance. Wall foundations subject to scour should be designed so that the nominal bearing resistance, in conjunction with the depth of scour determined for the check flood for scour, provides adequate resistance to support the unfactored Strength Limit State Loads with a resistance factor of 1.0.

B. Eccentricity

The overturning calculation used in ASD is replaced with the eccentricity check. Eccentricity of loading on walls founded on spread footings is identical to the requirements for abutments, should be calculated for each load group and checked to meet the following criteria:

- $E < B/3$ for foundations on soil
- $E < 0.45B$ for foundations on bedrock

C. Sliding

Sliding calculations for walls on spread footings are identical to the requirements for abutments described in 5.3.8 Sliding. Passive pressures in front of the wall should be neglected. To maximize the effect of the live load surcharge, the horizontal component of the live load surcharge should be included, whereas the vertical component over the heel or base should be neglected.

D. Pile Resistance

The design of walls founded on deep foundations is similar to the design requirements described in 5.4.1.12 Abutments Supported on Pile Foundations and 5.7 Piles.

E. Overall Stability

The overall global stability of retaining walls should always be checked at Service I load combination with a resistance factor, ϕ , of 0.65.

5.6.1.6 Service Limit State Checks

Service limit state wall settlement should be checked with the following performance limits in mind:

- Total settlement can be estimated using the procedures and criteria described in 5.3.6 Settlement. The tolerable total settlement criterion generally considers its effect on serviceability.
- Settlement may be critical where the wall interacts with other structures, e.g. at the approach to a pile supported abutment.
- Distortion, i.e., the ratio of horizontal movement to vertical movement should be less than 1/500.
- Lateral deformations will usually take place during construction and be affected by wall batter, compaction effort and construction equipment next to the wall.
- Global stability.

5.6.1.7 Design Considerations

All retaining walls should be designed with consideration of frost protection (Section 5.2.1), scour protection (Section 2.3.11), bearing resistance (Section 5.3.5), settlement (Section 5.3.6), stability (Section 5.3.7), drainage considerations (Section 5.3.11), and seismic considerations (Section 5.2.5), as appropriate.

All retaining walls require a subsurface investigation of the underlying soil or bedrock that will support the structure or tie-back elements. Minimum requirements for number, spacing and depth of exploratory borings are provided in Section 2.10 Subsurface Exploration Programs.

5.6.1.8 Aesthetics

Retaining walls should have a pleasing appearance that is compatible with the surrounding terrain and other structures in the vicinity. Aesthetic requirements include consideration of the wall face material, the top profile, the terminals, and the surface finish (texture, color, and pattern). Where appropriate, provide planting areas and irrigation conduits. In higher walls, variation in treatment is recommended for a pleasing appearance. High, continuous walls are generally not desirable from an aesthetic standpoint. Consider stepping high or long retaining walls in areas of high visibility.

5.6.2 Gravity Retaining Walls

Gravity retaining walls are generally trapezoidal in section and derive their capacity to resist lateral soil loads through a combination of self-weight and sliding resistance. Gravity walls can be subdivided into rigid gravity walls, which will be discussed in this section, MSE walls discussed in Section 5.6.5.4, and prefabricated proprietary walls discussed in Section 5.6.5.

5.6.2.1 Design Section

Gravity wingwalls should have a thickness at the top of 1'-6" in a direction normal to the front neat line. Batters on the front and back faces of wingwalls should be related to the vertical plane, which is normal to the front neat line. The front neat line is a horizontal line, which is the intersection of the top of footing elevation and the front face of the wall. If there is no footing, a working elevation should be used. Gravity walls of any length should be constructed to work integrally with abutments.

5.6.2.2 Earth Loads

Rigid gravity walls should be designed as unrestrained, which means that they are free to rotate at the top in an active state of earth pressure. An active earth pressure coefficient, K_a , should be calculated using Coulomb Theory as described in Section 3.6.5.1.

5.6.3 Gravity Cantilever-type Retaining Walls

This section discusses gravity, cantilever-type retaining walls. This type of wall is differentiated from a non-gravity cantilever retaining wall by relying on the bending action of the wall stem, in addition to self-weight, to resist lateral earth pressures. The footing contributes to the wall stability in overturning and sliding. Non-gravity cantilever retaining walls (i.e., sheet pile walls) are discussed in Section 5.6.4.

5.6.3.1 Design Section Gravity Cantilever Retaining Walls

Cantilever walls should have the following limits for wall thicknesses (heights are measured from top of the wall footing):

- 1'-3" minimum thickness for walls up to 6 feet high at the highest point.
- 1'-6" minimum thickness for walls between 6 feet and 20 feet in height at the highest point.
- 1'-9" minimum thickness for walls over 20 feet in height at the highest point.
- Walls should be increased in thickness to accommodate recessed architectural treatment, as necessary.

Wingwalls that are 15 feet or more in height at the ends may be designed with butterfly wings, if economical to do so.

On wingwalls that are less than 15 feet in height at the ends, the footing may be reduced in length if it is not required for structural or geotechnical considerations. The wall should be detailed with the bottom of the wall at the elevation of the top of the footing.

Tops of parapets should not have elevations above the adjacent curbs or sidewalks.

Gravity cantilever wingwalls more than about 20 feet long should be designed to work independently from the abutment, except that footings should be integral. A vertical contraction or expansion joint with no shear

key should be used near the corner between the abutment and the wingwall. The front face of the wingwalls should be recessed 2 inches back from the face of the wall on the abutment side of the contraction or expansion joint.

Gravity cantilever type wingwalls that are less than about 20 feet long should also be designed independently from the abutment; however, the wingwall should be restrained at the corner through an integral connection to the abutment. Soil pressure under the footing, sliding, and eccentricity should be evaluated as discussed in Section 5.3 Spread Footings. The restraining force at the corner is considered to be caused by at rest lateral earth pressure, as a minimum, because of the wingwall's inability to deflect at the corner. The corner should be designed to be restrained by concrete beam action with horizontal reinforcing steel anchored into the abutment section.

5.6.3.2 Earth Loads

For earth loads relative to cantilever walls refer to Section 3.6. Load factors for earth loads and surcharge loads are provided in Table 5-4. In the case of a long wall with a variable height, the wall should be divided into more than one design section. The design section should be at the highest third point of the wall. Refer to Figure 5-6 for further guidance.

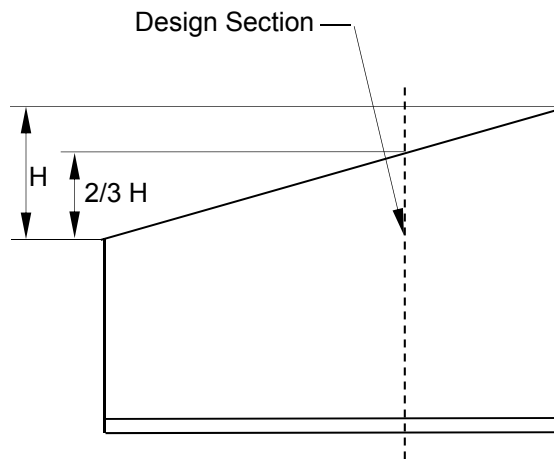


Figure 5-6 Retaining Wall Design Section

Gravity cantilever walls should be designed as unrestrained, which means that they are free to rotate at the top in an active state of earth pressure. An active earth pressure coefficient, K_a , should be as described in Section 3.6.4 and factored as specified in Table 5-6.

5.6.4 Non-Gravity Cantilever and Anchored Retaining Walls

This section discusses non-gravity cantilever retaining walls. Non-gravity cantilever retaining walls derive lateral resistance through embedment of vertical wall elements, sometimes in combination with anchors or tie-backs. These vertical elements may consist of sheet piles, soldier piles, caissons, or drilled shafts. The vertical elements may form the entire wall face or they may be spanned structurally using timber lagging or other materials to form the wall face.

The design of cantilever and anchored walls include additional checks for the geotechnical resistance of anchors in pullout, bearing resistance of vertical elements, the passive resistance of vertical elements and the structural capacity of anchors, vertical wall elements and wall facing. Resistance factors specific to cantilever and anchored walls can be found in LRFD Table 11.5.6-1

5.6.4.1 Soil Nail Walls

Soil nail walls are technically anchored walls that employ a reinforced soil mass serving as a gravity retaining structure. The reinforced soil mass of a soil nail wall is created by drilling and grouting steel anchors into an in-situ soil mass. The anchored soil mass is then covered with shotcrete. The temporary shotcrete face is then covered with a permanent facing system, typically cast-in-place concrete, precast concrete, or timber lagging. Soil nail walls are suited to cut situations only.

Soil nail walls are relatively low cost and can be used in areas of restricted overhead or lateral clearance. Soil nail walls are built from the top down and are only suitable if the site soils have adequate “stand-up” time of 1 to 2 days in a 5 foot vertical cut. Soil nail walls are not applicable to sites with bouldery soils, which could interfere with nail installation. This wall type is not recommended in uniform or water bearing sands or where there is a potential deep failure surface. Maximum wall heights of 30 feet are allowed.

These walls can be designed by the Designer or specified as a design-build item. The PS&E package should include the plan development information discussed in Section 5.6.5.5. Special Provisions have been developed for soil nail walls. Check with the Geotechnical Designer for the current Special Provision.

5.6.5 Prefabricated Proprietary Walls

Prefabricated proprietary walls are any prefabricated wall system approved by MaineDOT and produced by a manufacturer licensed by the wall vendor. Prefabricated proprietary walls are typically designed by the vendor, but may be designed by the Geotechnical Designer. In design, the vendor should

consider external stability with respect to sliding and overturning (at every module level) and internal stability with respect to pullout, as specified in LRFD 11.10 and Chapter 3, Loads. The Geotechnical Designer is required to verify acceptable global stability of the wall using a resistance factor of 0.65 prior to advertisement. The factored bearing resistance of the wall foundation soil or bedrock must be shown on the plans.

5.6.5.1 Proprietary Retaining Walls

Retaining walls available for a given project include standard walls, where the responsibility of the design is the Structural Designer, and proprietary walls, which are designed by a wall manufacturer. There are MaineDOT preapproved proprietary wall systems and non-approved proprietary wall systems. Preapproved wall systems have been extensively reviewed by MaineDOT and are listed on the MaineDOT Qualified Products List (QPL) webpage for the particular wall type. MaineDOT has developed a review process for the pre-approval of non-approved proprietary walls systems (MaineDOT, 2010), available on the MaineDOT QPL website. Non-approved proprietary walls must go through the pre-approval review process prior to use of the wall system.

5.6.5.2 Prefabricated Concrete Modular Gravity Walls

Prefabricated concrete modular gravity (PCMG) walls covered under Special Provision 635 should consist of either “T-Wall[®]” as provided by a licensed manufacturer of the Neel Company, Springfield, Virginia, or “DoubleWal[®]” as provided by a licensed manufacturer of DoubleWal Corp., Plainville, Connecticut.

PCMG walls should be designed in accordance with Special Provision 635 and LRFD Article 11.11. In general, the design requirements are similar to the requirements for conventional retaining walls and abutments, with the exception of pullout resistance requirements and dedicated analyses at each level of modular units.

PCMG walls should be considered on all projects where metal bin, gabion, MSE, and cast-in-place walls are considered. PCMG walls should be limited to a maximum height of 27.5 feet and a maximum batter of 1/6 (2 inches per foot). Refer to Section 5.6.5.5 PS&E for Project with Proprietary Walls for plan development requirements.

Whenever possible, a battered wall will be used in preference to a vertical wall. The use of a vertical wall design may be necessary where the wall is located on a horizontal curve that may result in construction conflicts, or where property costs or other right-of-way considerations dictate.

PCMG walls should be designed with adequate embedment for frost protection. Refer to Section 5.2.1 Frost for guidance.

PCMG walls should not be used in locations where there is scour potential, unless suitable scour protection can be economically provided. Refer to Section 2.3.11 Scour for guidance.

Where special drainage problems are encountered, such as seepage of water in the excavated backslope, underdrain will be provided behind the wall. Refer to Section 5.3.11 Drainage Considerations for further guidance.

Where PCMG walls will come in contact with salt water, all rebar should be epoxy coated and the concrete should be class LP. The appropriate note from Appendix D Standard Notes Prefabricated Concrete Modular Gravity Wall should be on the contract drawings.

Where PCMG walls are to be located in water, consideration should be given to drainage behind the wall. As a minimum, the Designer should consider a 12 inch thick layer of crushed stone extending vertically along the inside wall face. Crushed stone should be separated from surrounding soils with an erosion control geotextile. When drainage features are used for PCMG walls, payment should be considered incidental.

PCMG walls may be considered to retain soil supporting bridge substructures, with the exception of bridges over waterways. Their use is subject to the approval of the Assistant Bridge Program Manager at the PDR stage. These types of walls shall be designed for a service life of 100 years. The PCMG concrete shall contain a minimum of 5.5 gal/yd³ of corrosion inhibitor and use corrosion resistant reinforcing. PCMG walls which retaining abutments and are within 30 feet of the edge of a roadway or 50 feet of the centerline of a railway track should be designed for collision forces or protected with a crashworthy barrier (see 5.4.1.7.E). Additional design criteria for abutments retained by PCMG walls are similar to those for MSE walls described in 5.6.5.4.

Cofferdams required for PCMG wall construction should be considered incidental to wall construction. The appropriate notes from Appendix D Standard Notes Prefabricated Concrete Modular Gravity Wall should be on the contract drawings.

PCMG walls are measured and paid for by the area of wall face, as determined from the plan dimensions. The PCMG pay item includes compensation for excavation, excavation support foundation material, backfill material, and wall design. Consult Special Provision 635 for current measurement and payment information.

5.6.5.3 Precast Concrete Block Gravity Walls

Precast concrete block gravity walls consist of walls where precast concrete units are stacked vertically, function either as a gravity retaining wall or as a facing with geosynthetic-reinforced soil backfill, as covered in Special Provision 635. The connection between adjacent courses of modular blocks may be mechanical (cast knobs) or frictional. A preference is for mechanical connections. These wall systems are generally limited to a maximum height of 4.5 feet when the precast concrete units function as a gravity wall without reinforced backfill and no surcharge load is applied. When wall height is in excess of 4.5 feet or a surcharge is applied, geosynthetic reinforcement may be added to the modular blocks to create a geosynthetic-reinforced soil (GRS) wall.

Precast Concrete Block Gravity Walls without reinforced backfill should meet the design requirements of LRFD 11.11. If the backfill is reinforced, walls should meet the design requirements of LRFD 11.10 and BDG

5.6.5.4.B. Geosynthetic-Reinforced Soil Walls.

Blocks for modular block walls are made from wet cast concrete. Wall systems comprised of dry cast concrete are susceptible to degradation caused by freeze-thaw and are not an approved wall type. Precast concrete block gravity walls are not permitted in waterways.

5.6.5.4 MSE Walls

A. MSE Walls with Steel Reinforcement

This type of MSE wall uses galvanized strips or mats of steel to reinforce soil and create a reinforced soil block behind the wall face. The reinforced soil mass acts as a unit and resists the lateral loads through the dead weight of the reinforced mass. MSE walls are constructed from the bottom up and are therefore best suited for fill situations.

With a few exceptions, the procedure for the design of MSE walls using LRFD is identical to that followed using ASD. External stability evaluations include bearing resistance, sliding, and eccentricity. Internal stability calculations include pullout and rupture of reinforcements, capacity of reinforcement connections to the wall face, and structural capacity of the wall facing. MSE walls are typically designed by the wall manufacturer for internal and external stability. All MSE walls should be designed in accordance with:

1. LRFD Article 11.10

2. Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volumes I and II, November 2009, FHWA-NHI-10-024 and FHWA-NHI-10-025
3. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, November 2009, FHWA-NHI-09-087
4. Standard Specification Section 636 – Mechanically Stabilized Earth Retaining Wall

It is the responsibility of the Geotechnical Designer to assess the wall for bearing resistance, settlement, and global slope stability.

The calculation of lateral earth pressure on MSE walls should be as specified in AASHTO LRFD 3.11.5.8.

MSE walls with steel reinforcement and precast panels are relatively low in cost. These walls do require a high quality backfill with strict electrochemical requirements, as defined in the Standard Specifications Section 636 - Mechanically Stabilized Earth Retaining Wall. The base width of MSE walls is typically 70% of the wall height, which requires considerable excavation in a cut situation. Therefore, in a cut situation, base width requirements usually make MSE structures uneconomical and difficult to construct. It is best to limit the height to approximately 35 feet for routine projects.

Facing options depend on the aesthetic and structural needs of the wall system. Facing options typically include precast modular panels with various shapes and texturing options. The facing type used can affect the ability of the wall to tolerate settlement, depending on whether continuous vertical joints between adjacent panels are specified. Refer to Section 5.6.1.8 Aesthetics for further guidance.

MSE walls are inherently flexible and can tolerate moderate settlements without suffering structural damage, depending upon the MSE wall panel shape and alignment.

MSE walls are not appropriate if very weak soils are present that will not support the wall and that are too deep to be over excavated, or if a deep failure surface is present that result in slope instability. In these cases, a deep foundation or soil modification may be considered.

MSE walls may be used to retain soil supporting bridge substructures. The substructure units may be either spread footings or pile supported, with the following additional design criteria:

- The MSE wall shall be designed to provide a service life of not less than 100 years.
- For the analysis of spread footings on top of the reinforced soil zone, a factored bearing resistance of 7 ksf should be used for the strength limit state, and a factored bearing resistance of 4 ksf should be used to limit settlements to less than approximately 0.5 inch.
- A minimum distance of 4 feet should be provided between the bottom of the superstructure and the berm in front of the abutment breastwall or pile cap and behind the MSE top panel, for future bridge inspection and maintenance purposes.
- The minimum distance from the centerline of the bearing on the bearing on the abutment to the outer edge of the MSE wall facing should be 3.5 feet.
- A minimum distance of 2 feet should be provided between the back of MSE wall panels and the front face of abutment or pile cap.
- If the abutment is supported on piles or piles installed in sleeves, a minimum distance of 2 feet should be provided to allow compaction equipment to be used between piles or sleeves and the back face of panels, and to allow a 15° reinforcing strap skew to clear a typical 2-ft diameter pile sleeve.
- The top of the MSE panel in front of footings or pile caps should be set 1 foot above the berm elevation.
- If embedding spread footings for frost protection within the reinforced mass is impractical, provide at least 2 feet of soil cover and place the footing on a minimum 3-foot thick bed of compacted coarse aggregate.
- An impervious geomembrane consisting of low-permeability, 2-sided textured HDPE a minimum of 60 mils thick should be installed near the top of the reinforced soil zone to reduce the chance of water and salt-laden water infiltration into the reinforced backfill. The membrane should be bonded to the back face of the abutment, and sloped to shed water that infiltrates from the road surface.
- The need for fencing along the top of the wall should be investigated on a project by project basis.

Prior to selection of MSE walls for a project, consideration should be given to the location of any utility behind or within the reinforced soil backfill zone. It is best not to place utilities within the reinforced backfill zone because it would be impossible to access the utility from the ground surface without cutting through the soil reinforcement layers, thereby compromising the integrity of the wall. Coordination of the wall with project elements (such as drainage, utilities, luminaries, guardrail, or bridge elements) is critical to avoid costly change orders during construction. Moreover, failure of a sewer or water main located within an MSE wall mass could result in failure of the wall. As a result, MSE walls must not be used in areas where water and/or sewer utilities are present. It is also best to locate drainage features and signal or sign foundations outside of the MSE reinforced backfill zone.

Since MSE walls are proprietary and the wall vendor performs the design, it is imperative that the design requirements be clearly stated on the plans. If there are any unusual aesthetic requirements, design acceptance requirements, or loading conditions for which the wall needs to be designed, they should be clearly shown on the plans. Refer to Section 5.6.5.5 PS&E for Project with Proprietary Walls for plan development requirements.

MSE walls are measured and paid for by the area of wall face, as determined from the approved shop drawings. The high quality backfill and wall design are included in the MSE wall pay item. The Designer should consider this when comparing the cost of MSE walls with other wall systems, which typically pay for backfill as a separate pay item. Excavation is also paid for separately as common excavation. The Designer should consult the current Special Provision for measurement and payment information.

B. Geosynthetic-Reinforced Soil Walls

Geosynthetic-reinforced soil (GRS) walls are MSE or Precast Gravity Block walls with geosynthetic (polymeric) soil reinforcement. GRS walls are designed to create a reinforced soil volume behind a wall facing. Facing options include precast concrete modular panels or modular concrete blocks. Geosynthetic facings, although available, are not acceptable for permanent facing due to potential facing degradation when exposed to sunlight. Facings consisting of dry-cast concrete are susceptible to degradation caused by freeze-thaw and are not allowed. GRS walls are not permitted in waterways.

GRS walls are constructed from the bottom up and are therefore best suited for fill situations. The base width of GRS walls is typically 70% of the wall height, which requires considerable excavation in a cut situation.

It is best to limit the height of GRS walls to 20 feet or less for routine projects.

GRS walls have a low cost and can handle significant settlement. Compared to steel-reinforced systems, internal wall deformations may be greater and electrochemical backfill requirements less strict, but a high quality backfill is still required. Only geosynthetic products for which long-term product durability is well defined per LRFD 11.10.6.4 will be allowed.

GRS walls are proprietary and are designed by a wall manufacturer for internal and external stability. GRS walls shall be designed with a service life of not less than 75 years. The walls shall be designed in accordance with the following:

1. LRFD Article 11.10
2. Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volumes I and II, November 2009, FHWA-NHI-10-024 and FHWA-NHI-10-025
3. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, November 2009, FHWA-NHI-00-087

It is the responsibility of the Geotechnical Designer to assess the wall for bearing capacity, settlement, and global slope stability.

Since these preapproved walls are proprietary and the wall vendor performs the design, it is imperative that the design requirements for GRS wall be clearly stated on the plans. If there are any unusual aesthetic requirements, design acceptance requirements, or loading conditions or pressures for which the wall needs to be designed, they should be clearly shown on the plans. Refer to Section 5.6.5.5 PS&E for Project with Proprietary Walls for plan development requirements.

Coordination of the wall with project elements (such as drainage, utilities, luminaries, guardrail, or bridge elements) is critical to avoid costly change orders during construction. It is best to locate drainage structures and signal or sign foundations outside of the reinforced backfill zone.

5.6.5.5 PS&E for Project with Proprietary Walls

The PS&E package for a bridge project including proprietary wall item will include the following:

- General wall plan
- Wall profile, showing neat line top and bottom of the wall and final ground line in front of and in back of the wall
- Profiles showing the existing and final grades
- Typical wall cross section with generic details including batter
- Factored bearing resistance
- Foundation embedment criteria
- Leveling pad details
- General details for any desired appurtenances, such as coping or drainage requirements
- Project specific loads for other design acceptance requirements (examples: seismic loads, earth loads due to thermal movement of abutments)
- Special facing treatment (shape, texturing, color)
- Project-specific construction requirements (example: crushed stone)
- Highway approach cross sections showing only the face of the wall and footing

5.6.6 Geosynthetic Reinforced Soil Integrated Bridge Systems

GRS walls associated with Geosynthetic Reinforced Soil Integrated Bridge Systems (GRS-IBS) designed in accordance GRS-IBS Interim Implementation Guide, FHWA-HR-11-026, January 2011, may be considered for some bridges over waterways, with the approval of the Assistant Bridge Program Manager.

5.6.7 Anchored Wall Systems

5.6.7.1 CON/SPAN[®] Wingwall

CON/SPAN[®] wingwall systems may only be used in conjunction with CON/SPAN[®] precast drainage structures. The system consists of a precast face panel with a precast concrete soil anchor located near the base of the face panel. The wingwall system is connected to the CON/SPAN[®] drainage structure. The wall should be backfilled with

granular borrow material suitable for underwater backfill and compacted per the Standard Specifications. The maximum wall height available is 16.5 feet, and should only be used with a level backfill surface and seismic loads less than $a = 0.1g$ when a seismic analysis is required for design (ASCE, 2001). Refer to Section 3.7.2 Seismic Analysis for guidance.

The CON/SPAN[®] wingwall system should be designed in accordance with LRFD 11.9 Anchored Walls. The design requirements for the CON/SPAN[®] wingwall system should be included with the contract documents in Special Provision 534.

CON/SPAN[®] wingwall system should be placed on a footing, which serves both as a leveling slab and a structural foundation. This may include, but is not limited to a cast-in-place concrete footing, cast-in-place stub wall with footing, or a precast concrete footing meeting the requirements of Section 5.2.1 Frost, Section 5.3 Spread Footings, and Section 2.3.11 Scour. The footing should be sized to support the weight of the wall panels and weight of soil in and above the anchor system (ASCE, 2001).

The CON/SPAN[®] wingwall system should be equipped with a drainage system, consisting of a perforated drainage pipe installed in the backfill behind the wall, which outlets through a 4 inch diameter weep hole cast in the facing panel, per the manufacturer's requirements (ASCE, 2001).

5.6.7.2 Metal Structural Plate Headwall/Wingwall

Metal structural plate headwall/wingwall may only be used in conjunction with metal structural plate box culverts. However, preference should be given to the use of a PCMG wall system for increased durability. The headwall system consists of a metal structural plate face, which is connected to the top of the metal structural plate box with an anchor rod. The wingwall system consists of a metal structural plate face with a deadman connected to the face with a tie rod and whale system. The maximum wall height available is 14.25 feet.

The metal structural plate headwall/wingwall system should be designed in accordance with the most recent version of AASHTO LRFD. The design requirements for the metal structural plate headwall/wingwall system should be included with the contract documents.

5.6.8 Gabions

Gabion walls consist of stacked 3 feet cubed wire baskets, which are filled with stone. Groups of filled gabion baskets are stacked to construct a gravity wall. Gabion walls should be designed as specified in Section 3.6.7.2 Prefabricated Modular Walls. In designing gabion walls, a unit weight, γ , of 100 lb/ft³ should

be used for the weight of stone inside the baskets. Gabion walls should be backfilled with granular or gravel borrow. An angle of wall friction, δ , of 24° should be used for design. Wire for gabion baskets should be either PVC-coated or galvanized. A PVC coating is preferred as it does not flake off.

MaineDOT experience has shown that constructing gabion walls correctly can be costly and time-consuming. Disadvantages in the use of gabions include subjection to corrosion when placed in water and occurrence of vandalism by the cutting of the basket wires. Gabion walls should be used only in non-critical situations, in dry environments, and in rural areas, where the probability of corrosion and vandalism are less (MaineDOT, 2002). Gabion wall heights in excess of 6 feet are not recommended.

5.7 Piles

5.7.1 General

Piles should be considered when spread footings cannot be founded on bedrock or on competent soils at a reasonable cost. Piles should also be considered where soil conditions permit use of spread footings, but where the soils are susceptible to scour, liquefaction or lateral spreading.

Pile foundations should be designed so that the available factored geotechnical and drivability resistance is greater than the factored loads applied to the pile at the strength limit state. Service limit state design of driven pile foundations includes an evaluation of settlement, overall stability, lateral squeeze and lateral movement.

5.7.2 H-Piles

H-Piles used for bridge foundations should be comprised of rolled-steel sections of ASTM A572, Grade 50 steel, with a minimum yield stress of 50 ksi. Refer to Section 7.2.1 Structural Steel for H-pile material requirements.

5.7.2.1 Axial Resistance

The maximum factored axial design load applied to H-pile sections should not exceed the lesser of the factored structural pile resistance, the factored geotechnical pile resistance and the factored drivability resistance. The factored structural resistance of H-pile sections should be determined using a resistance factor, ϕ , listed below:

- $\Phi_c = 0.50$ for axial resistance of piles in compression and subject to damage due to severe driving where use of a pile tip is necessary.

- $\Phi_c = 0.60$ for axial resistance of piles in compression under good driving conditions, where use of a driving tip is not necessary.

For combined axial and flexural resistance of undamaged pile, the resistance factors are listed below:

- $\Phi_c = 0.70$ for axial resistance of H-piles in compression.
- $\Phi_f = 1.00$ for flexural resistance of H-piles.

The resistance factors, Φ_c and Φ_f , are to be used in interaction equations in LRFD 6.9.2.2.

The factored axial structural axial resistances of selected H-Pile sections are presented in Table 5-7. For the purposes of Table 5-7, the H-piles were assumed fully braced, and an effective length factor (K) of 1.0 was used. The Structural Designer should recalculate structural resistances for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific LPILE[®] analyses and recalculate structural resistances. For preliminary design purposes, however, the resistances provided in Table 5-7 may be used to estimate the factored structural axial resistance of that portion of the pile which is theoretically in pure compression, i.e., that portion below the point of fixity.

Commentary: Experience in using 50 ksi steel for H-Pile foundations has shown that the factored axial geotechnical resistance frequently governs design. This is particularly apparent for end-bearing piles on poor-quality and/or soft bedrock and for friction piles.

Table 5-7 Factored Axial Structural Resistance of Selected H-Pile Sections **$F_y = 50$ ksi and fully braced**

Pile Section	Factored Axial Structural Resistance	
	Good driving conditions $\Phi = 0.60$ (kips)	Severe driving conditions $\Phi = 0.50$ (kips)
HP 10x42+	372	310
HP 10x57	504	420
HP 12x53+	465	388
HP 12x63	552	460
HP 12x74	654	545
HP 12x84	738	615
HP 14x73+	642	535
HP 14x89+	783	653
HP 14x102	900	750
HP 14x117	1032	860

Note: Those marked + are preferred sections

The factored geotechnical and drivability resistances should be determined for site-specific conditions by the Geotechnical Designer. Consideration should be given to downdrag, soil relaxation, soil setup, lateral spreading and any other site-specific factors, which may affect the pile capacity during and after construction. The factored geotechnical resistance should be determined by applying a resistance factor which is dependent on the design method.

5.7.2.2 Lateral Pile Resistance for the Service Limit State

Horizontal movement of pile groups induced by lateral loads shall be evaluated for Service Limit State Design. The lateral resistance of a pile is governed by the loading condition, pile stiffness, stiffness of the soil, and the degree of fixity. The lateral resistance (P_L) and depth to fixity (D_f), for service limit state design for selected H-Pile sections in sand and clay are presented in Table 5-8 and Table 5-9, respectively. The factored lateral resistances presented in Tables 5-8 and 5-9 assume a resistance factor of 1.0 and a maximum lateral deflection of 1/8 inch.

Commentary: The lateral resistance and depth to fixity presented in Tables 5-8 and Table 5-9 were determined using the computer program LPILE[®] Plus Version 4, the soil properties stated, a fixed condition at the pile head, an infinitely long pile, an applied axial load equal to $A_s \times 0.25 \times F_y$ and a deflection of 1/8".

Table 5-8 Factored Lateral Resistance and Depth to Fixity for Strength Limit State Design for H-Pile Sections in Sand, $\phi=1.0$

Pile Section	Loose		Medium Dense		Dense	
	P _L (kips)	D _f (ft)	P _L (kips)	D _f (ft)	P _L (kips)	D _f (ft)
HP 10x42+	6.2	24	9.9	20	11.7	18
HP 10x57	7.1	26	11.4	22	13.6	19
HP 12x53+	8.1	28	13.3	24	16.1	20
HP 12x63	8.9	30	14.4	25	17.4	21
HP 12x74	9.4	31	15.6	25	18.9	22
HP 13x60	9.0	31	15.0	25	18.2	21
HP 13x73	9.8	32	16.4	26	20.0	22
HP 13x87	10.6	32	17.7	26	21.7	23
HP 14x73+	10.5	32	17.8	26	21.9	23
HP 14x89+	11.4	33	19.5	27	24.1	24
HP 14x102	12.3	35	20.9	28	25.9	25
HP 14x117	13.1	36	22.3	29	27.0	25

Note: Those marked + are preferred sections. P_L and D_f are determined assuming a friction angle, ϕ , of 32°.

Where the applied lateral load from the Service Limit State Load Combination exceeds that presented in Tables 5-8 and 5-9, or the pile length is less than the depth to fixity shown in the table, a more thorough analysis is recommended, using actual loading and soil conditions. Where soils differ from the conditions assumed in the tables, the Designer should complete a more thorough analysis.

Tables 5-8 and 5-9 present the lateral resistance and depth to fixity for a lateral load applied perpendicular to the pile flange. For conventional abutments and mass piers, H-piles should be oriented with the flange perpendicular to the substructure axis in the direction of the maximum applied lateral load. For conventional abutments and mass piers, where H-piles are oriented with the web perpendicular to the maximum applied lateral load, a thorough analysis of the foundation is recommended, using actual loading and soil conditions (Tables 5-8 and 5-9 do not apply). For integral abutments where the web is oriented perpendicular to the principal axis, the design should be in accordance with Section 5.4.2 Integral Abutments.

Table 5-9 Factored Lateral Resistance and Depth to Fixity for Service Limit State Design for H-Pile Sections in Clay, $\phi=1.0$, Load Perpendicular to Flange

Pile Section	Soft ¹		Medium Stiff ²		Stiff ³	
	P _L (kips)	D _f (ft)	P _L (kips)	D _f (ft)	P _L (kips)	D _f (ft)
HP 10x42+	5.1	22	9.2	18	13.1	16
HP 10x57	5.5	24	10.2	20	14.5	18
HP 12x53+	6.3	26	11.7	21	16.6	19
HP 12x63	6.7	27	12.4	22	17.6	19
HP 12x74	7.1	27	13.1	22	18.7	20
HP 13x60	7.0	27	12.8	22	18.2	19
HP 13x73	7.5	28	13.8	23	19.5	21
HP 13x87	7.9	29	15.6	25	20.7	21
HP 14x73+	8.1	29	14.8	24	21.0	21
HP 14x89+	8.7	31	15.9	25	22.5	22
HP 14x102	9.1	31	16.7	26	23.6	22
HP 14x117	9.5	32	17.5	26	24.8	24

Note: Those marked + are preferred sections.

¹S_u = 375 psf, ²S_u = 750 psf, ³S_u = 1125 psf

5.7.3 *Layout and Construction*

The pile spacing should not be larger than is reasonable or practical. The center-to-center pile spacing should not be less than 30 inches or 2.5 to 3 times the pile diameter. A reasonable maximum spacing for piles in the back row of abutments is 12 feet.

Care should be exercised in locating piles to avoid interference with other piles, both in the final position and during the driving process. If a plumb pile in the back row is located directly behind a battered pile in the front row, the Contractor may be forced to plan his sequence of pile driving and cut-offs in a less efficient manner than if the back row of piles were staggered with the front row.

The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 inches. The tops of piles should project at least 18 inches into the pile cap after all damaged pile material has been removed.

All piles should be equipped with a driving shoe. Refer to Standard Specification Section 501 – Foundation Piles for further guidance.

5.7.4 *Concrete Piles*

Concrete piles are used as displacement piles provided they can be driven without damage, that is, there are no boulders or hard driving dense soils. Two types of concrete piles are precast conventionally reinforced and precast prestressed piles. Both types are of constant cross section, though they may have tapered tips. Pile shapes include square, octagonal, and round sections and may be either solid or hollow. Typical pile cross sections used range from 10 inches to 16 inches, but sizes above and below this range are also produced. Refer to LRFD Article 5.13.4, Concrete Piles, and FHWA, 1998 for detailed information regarding concrete piles.

Precast concrete piles are suitable for use as friction piles when driven in sand, gravel, or clays. Precast concrete piles are capable of high capacities when used as end bearing piles. In boulder conditions, a short piece of structural H-pile section or “stinger” may be cast into or attached to the pile for penetration through the zone of cobbles and boulders.

Conventionally reinforced concrete piles (concrete with reinforcing steel bars and spiral reinforcing steel cages) are susceptible to damage by mishandling or driving. Prestressed concrete piles are more vulnerable to damage from striking hard layers of soil or obstructions during driving than conventionally reinforced concrete piles. Piles should be equipped with a metal driving shoe for hard driving conditions. High stresses during driving can cause cracking in all concrete piles.

Precast piles are difficult to splice, particularly prestressed piles. Accurate knowledge of pile lengths is required when using concrete piles, as they are also difficult to shorten. Special precautions should be taken when placing concrete piles during cold weather. Temperature gradients can cause concrete to crack due to non-uniform shrinkage and expansion.

A concrete pile foundation design should consider that deterioration of concrete piles can occur due to sulfates in soil, ground water, or sea water; chlorides in soils and chemical wastes; or acidic ground water and organic acids. Laboratory testing of soil and ground water samples for sulfates and pH is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and ground water samples is recommended when chemical wastes are suspected.

5.7.5 *Steel Pipe Piles*

5.7.5.1 Design - General

The maximum factored applied axial load on any pipe pile shall not exceed the lesser of the factored structural compressive resistance, the factored axial geotechnical resistance and the factored drivability pile resistance. For the strength limit state, the factored axial compressive structural resistance of pipe piles (P_r) shall be estimated using the following resistance factors (Φ_c):

- $\Phi_c = 0.60$ for piles subject to damage in severe driving conditions where use of a pile tip is necessary
- $\Phi_c = 0.70$ for piles under good driving conditions where use of a pile tip is not necessary

The nominal compressive structural resistance (P_n) for pipe piles loaded in compression should be estimated as specified in LRFD 6.9.5.1 using the column slenderness factor, λ .

At the strength limit state an axial resistance factor, Φ_c , of 0.80, and a flexural resistance factor, Φ_f , of 1.0 should be applied to combined nominal axial and flexural resistance in the interaction equation in LRFD 6.9.2.2.

5.7.5.2 Material and Design Section

Pipe piles consist of seamless, straight butt-welded or spiral butt-welded metal shells. Steel pipe piles may be driven in groups, to support ground-level pile caps, or in-line to form pile bents. They are available in a wide range of diameters. Typical wall thicknesses are limited to the range of

1/2" to 1 inch. MaineDOT practice has commonly limited their use to 24 to 32 inch diameters when used in pier bents. All pipe piles are filled with Class A concrete after driving. Additionally, pipe piles employed as pier bents are internally reinforced with a reinforcing cage.

Concrete filled pipe piles have a high load-carrying capacity and provide high bending resistance where an unsupported length is subject to lateral loads. For design criteria and corrosion protection of pipe piles in pier bents, refer to Section 5.5.2.5 Pile Protection and 5.5.2.6 Pipe Pile Coatings and Cathodic Protection.

Pipe piles may be driven open or closed ended. If the capacity from the full pile toe is required, the pile should be driven closed ended, with a flat plate or conical tip. Closed ended types are preferred, except if the pile is designed as a friction displacement pile.

If obstructions are expected, the pile should be open-ended, so that it can be cleaned out and driven further. Open-ended piles driven in sands or clays will form a soil plug at some stage during driving. At this stage, the pile acts like a closed ended pile and can significantly increase the pile toe resistance. Piles driven open-ended should be cleaned, leaving a length of soil plug ranging from two to three pile diameters, and filled with concrete after driving.

Steel pile material should conform to ASTM A252 Grade 3. Open-ended piles should be reinforced with steel cutting shoes to provide protection against damage. When pipe piles are driven to weathered bedrock or through boulders, an end plate or conical point with a rounded nose is often used to prevent distortion of the pile nose. End closures should be cast steel, conforming to the requirements of ASTM A27 (grade 65-35) or ASTM A148 (grade 90-60).

For high vertical or lateral loads, open-ended pipe piles may be socketed in bedrock. They can also have a structural shape such as an H-section inserted into the concrete and socked into bedrock. Anchoring pipe piles with rock dowels or anchors is not recommended and should only be considered when the preceding alternatives are found to be not feasible.

Pipe piles can be spliced using full penetration groove welds or proprietary splicing sleeves that provide full strength in bending.

5.7.6 Downdrag

Where the soil deposit in which piles are installed is subject to settlement, downdrag forces may be induced on piles. As little as 1/2" of differential settlement may induce downdrag forces. Downdrag loads reduce the usable

pile capacity. Possible development of downdrag loads on piles should be considered when:

- Sites are underlain by compressible clays, silts, or peats
- Fill has been recently placed on the surface
- The groundwater has been substantially lowered

Downdrag loads should be considered as permanent additional axial loads when the nominal bearing capacity of the pile foundation is evaluated, and when settlement of the pile foundation is evaluated.

To calculate downdrag loads on piles, the traditional approach is the total stress α -method, which is used for computing downdrag in cohesive soils. Newer methods are based on the relationship between pile movement and negative shaft resistance, and described in Briaud and Tucker (1993). The downdrag loads should be factored by the appropriate load factor for downdrag, γ_p , and added to the factored vertical dead load applied to the pile.

If downdrag forces are significant, they can be reduced by applying a thin coat of bitumen to the pile surface (Dixon, et. al., May 1998). Battered piles should be avoided where downdrag loads are expected due to induced bending moments in response to settlement. These bending moments can result in pile deformation. In situations where downdrag forces cannot be reduced by applying bitumen coating, the Designer should consider:

- Forcing soil settlement prior to driving piles by preloading and consolidation the soils
- Using lightweight fills
- Increasing the pile size
- Sleeve piles

5.7.7 Pile Installation Quality Control and Nominal Pile Resistance

The nominal resistance a pile is driven to in the field is a function of the level of quality assurance/control provided during construction operations. The resistance factors for nominal pile resistances are presented in Table 5-10. These resistance factors are based upon construction quality control beyond the standard subsurface exploration and static pile capacity analysis.

Table 5-10 Resistance Factors for Driven Piles

Construction Control Method	Resistance Factor, Φ_{dyn}
Static load test of at least one pile, with dynamic testing of at least 2% to 5% of the production piles.	0.80
Dynamic testing with signal matching of at least 1 pile per substructure, but no less than 2 dynamic pile tests from opposite corners for substructures longer than 40 feet or with more than 15 piles, but no less than 2% of the production piles at any one site, and up to 5% of the production piles for sites with moderate to highly variable subsurface conditions.	0.65
Wave equation analysis without dynamic measurements or load test	0.40

A pile group is classified as nonredundant if there are less than five (5) piles in the group. If a pile group is nonredundant, past LRFD practice dictated a 20 percent reduction of the pile resistance factors, Φ_{dyn} , provided in Table 5-10, and should be considered to provide a uniform level of safety.

Pile testing programs should include, at a minimum, wave equation analyses. Wave equation analyses confirm that the design pile section can be installed to the desired depth and ultimate capacity, without exceeding allowable pile driving stresses, with an appropriate driving system and criteria.

In addition to wave equation analyses, pile testing programs should also include dynamic load tests or, rarely, static load tests. Dynamic testing with signal matching should be considered in order to:

- Field-verify the nominal pile axial resistance
- Establish driving criteria
- Monitor piles installed in difficult subsurface conditions, such as soils with obstructions and boulders, or a steeply sloping bedrock surface
- Verify consistent hammer operation during extended pile installation operations
- Justify higher resistance factors

In general, the pile testing program should be commensurate with the design assumptions; for example, at least 1 pile per bearing stratum will be tested.

Pile testing programs should specify the number, location, and time of all dynamic tests and/or static pile tests. When a dynamic load test program is specified, the following requirements shall apply:

- For large pile groups with more than 20 piles, the first and second pile tests shall be conducted at opposite corners of the substructure, and at least one additional dynamic test shall be conducted mid-production, after approximately one half of the production piles have been installed.
- Post-driving analyses (CAPWAP) are required.
- Provisions for 24 to 72 hour pile restrikes shall be included, for substructures where setup or relaxation effects are expected.
- Provisions for 24 to 72 hour dynamic restrike tests are mandatory for friction piles or piles designed to end bear in any strata other than bedrock.
- Provisions should be provided for the conduct of additional dynamic load tests during production, for field verification that the driving criteria are consistently achieving the required nominal pile resistances.

A minimum of 2% of the piles shall be tested when dynamic (or static) testing is specified. It may be necessary to test 5% or more piles, when there are more than 20 piles in a substructure, when difficult driving is expected, when variable or inconsistent soil conditions are expected, or when additional tests during production are necessary to verify hammer performance and geotechnical resistances.

The establishment of the driving criteria should include limiting driving stresses to the following thresholds:

- For steel piles in compression and tension, driving stresses should not exceed 90% of the yield strength of the pile material. For 50 ksi steel, this results in a maximum driving stress of 45 ksi.
- For concrete filled pipe piles, if unfilled when driven, driving stresses should not exceed 90% of the yield strength of the steel shell material.
- For concrete piles, driving compressive stresses should not exceed 0.85 times the concrete compressive strength. Tensile stresses during driving should not exceed 0.70 times the yield strength of the steel reinforcement.

- For prestressed concrete piles, driving compressive stresses should not exceed 0.85 times the concrete compressive strength minus the effective prestress. Tensile stresses during driving should be limited to 0.095 times the square root of the compressive strength (ksi) plus the effective prestress.

5.8 Drilled Shafts

Drilled shafts may be an economical alternative to spread footings or pile foundations. Drilled shafts can be an advantageous foundation alternative when:

- Spread footings cannot be founded on suitable soil, or bedrock, within a reasonable depth or when driven piles are not viable.
- Traditional piles would result in insufficient embedment depth and rock-socketed deep foundations are needed.
- Scour depth is large.
- Foundations are required in stream channels. Drilled shafts will avoid expensive construction of cofferdams. Advantages are the reduction of the quantities and cost of excavating, dewatering, and sheeting, and in limiting environmental impact.
- The elimination of waterline footings is advantageous and possible by extending drilled shafts as a column up to the pier cap.
- The foundation is required to resist high lateral loads or uplift loads.
- There is little tolerance for deformation.
- The cost and constructability of seals and caps for pile supported structures is high.

Although there are many references for the design and analysis of drilled shafts, MaineDOT follows the procedures found in FHWA, 2010 and LRFD Article 10.8.

The structural design of drilled shafts is similar to the LRFD method for a column with axial load and bending, and shear. Interaction diagrams should be developed to assess resistance to combined axial and bending.

The Bridge Program has developed a Special Provision to govern the construction of drilled shafts. Consult the Geotechnical Designer for the current version.

5.9 Embankment Issues

Embankment design considerations include settlement, slope stability, and bearing capacity at the base. Special design requirements for embankments will be presented in the Geotechnical Report. The Geotechnical Designer should review plans to determine any special design requirements with regard to an embankment.

5.9.1 *Embankment Settlement*

The embankment settlement should be evaluated using the methods discussed in Section 5.3.6 Settlement and must be within tolerable limits. Differential settlement is more of a concern than total settlement and should be evaluated by the Geotechnical Designer. Tolerable settlement also depends upon the structural integrity of the bridge or culvert and should be coordinated with the Structural Designer.

If settlement exceeds the tolerable limits, or the time needed to allow for settlement is excessive, several methods to address this are available to the Designer:

- Compressible materials can be removed and replaced to limit settlements.
- Preloads alone or in combination with surcharge can be used to complete settlements prior to construction.
- Prefabricated vertical drains can be used in conjunction with preloads to accelerate settlements.
- Lightweight fill materials such as tire shreds, geofoam or light weight concrete fill can be used.

The use of a preload, surcharge, or prefabricated vertical drains should be accompanied by the use of instrumentation (settlement platforms, piezometers, inclinometers) to assist in determining that an acceptable level of consolidation has taken place.

5.9.2 *Embankment Stability*

Embankment stability problems most often occur where embankments are to be built over soft weak soils such as low strength clays, silt, or peats. There are three major types of instability that should be considered in the design of embankments over weak foundation soils: circular arc failure, sliding block failure, and lateral squeeze. These stability problems are defined as “external” stability problems. “Internal” stability problems generally result from the

selection of poor quality materials and/or improper placement requirements. Refer to Section 5.3.7 Overall Stability for methods of analysis.

Once the soil profile, soil strengths, and depth of water table have been determined by both field explorations and field and laboratory testing, the stability of the embankment can be analyzed. The evaluation of slope stability of earth slopes with or without a foundation unit should be investigated at the Service I Load Combination and an appropriate resistance factor. The resistance factor, ϕ , may be taken as:

- 0.75 - where the geotechnical parameters are well defined, and the slope does not support or contain a structural element
- 0.65 – where the geotechnical parameters are based on limited information or the slope contains or supports a structural element.

Available slope stability programs produce a single factor of safety. In light of this, the past practice of checking overall slope stability using ASD methods may be continued to insure that slopes and slopes with footings have a factor of safety equivalent to 1.3 and 1.5, respectively.

If the load and resistance balance cannot be met, several methods to improve stability can be undertaken:

- Removal and replacement of the weak material
- Use of a mid-slope berm or other variations of berms
- Soil reinforcement with steel, geogrid, or geotextile
- Installation of prefabricated vertical (wick) drains, sand drains, or stone columns
- Instrumentation and control of embankment construction
- Installation of a structural support such as a retaining wall

Lateral squeeze can occur when the lateral movement (consolidation) of soft soils transmits an excessive lateral thrust, which may bend or push an adjacent substructure. The best way to minimize lateral squeeze is to complete embankment settlements prior to construction of adjacent substructures.

5.9.3 Embankment Bearing Capacity

The embankment bearing resistance should be evaluated using the methods discussed in Section 5.3.5 Bearing Resistance. The factored bearing resistance should equal or exceed the factor applied loads.

5.9.4 Embankment Seismic Considerations

Currently, there are no LRFD codes for embankment seismic design. Therefore, using allowable stress design methods, a minimum seismic factor of safety of 1.0 is acceptable for slope stability and liquefaction. Refer to Section 3.7.4 Embankment & Embankments Supporting Substructure Units. Should poor seismic performance of an embankment impact the overall serviceability or performance of a critical structure the Department may specify a higher level of seismic performance or specify appropriate seismic provisions.

If the seismic slope stability factor of safety falls below 1.0 using the seismic coefficient-factor of safety method, a permanent seismic deformation analysis should be conducted using the Newmark Method (Newmark, 1965). This method approximates the cumulative vertical deformation or settlement at the back of the slope for a given earthquake ground motion. The failure mass is modeled as a block on a plane. A maximum allowable seismic settlement of 6 inches at a bridge approach, resulting from the design earthquake event, is considered acceptable. Refer to Section 3.7 Seismic for loading considerations.

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Chapter 6

CONCRETE



Hancock-Sullivan Bridge, Hancock-Sullivan



Ogunquit Beach Bridge, Ogunquit

6	CONCRETE.....	6-1
6.1	Precast Concrete.....	6-1
6.1.1	General.....	6-1
6.1.1.1	Prestressing	6-1
6.1.1.2	Camber, Deflection, and Blocking	6-1
6.1.1.3	Section Properties	6-2
6.1.1.4	Constructibility Check.....	6-2
6.1.1.5	Allowable Stresses at Service Loads	6-2
6.1.2	Materials.....	6-2
6.1.2.1	Concrete.....	6-2
6.1.2.2	Prestressing Strand.....	6-3
6.1.2.3	Mild Reinforcement	6-3
6.1.3	Economy	6-3
6.1.3.1	Release Strength.....	6-3
6.1.3.2	Beam Sections	6-3
6.1.4	Design Requirements.....	6-4
6.1.4.1	Concrete Cover	6-4
6.1.4.2	Voided Slab and Butted Box Beam Bridges	6-4
6.1.4.3	NEBT, AASHTO I-Girder, and Spread Box Beam Bridges	6-6
6.2	Cast-In-Place Concrete	6-6
6.2.1	Materials.....	6-6
6.2.1.1	Concrete.....	6-6
6.2.1.2	Reinforcing Steel.....	6-8
6.2.2	Decks	6-9
6.2.2.1	Standard Design of Concrete Slab on Steel Girders	6-10
6.2.2.2	Standard Design of Concrete Slab on Concrete Girders.....	6-12
6.2.2.3	Precast Deck Panels	6-13
6.2.3	Drilled and Anchored Bolts/Bars.....	6-15
6.2.3.1	Specification Procedures.....	6-16
	References	6-20
	Table 6-1 Post Tension and Diaphragm Locations	6-4
	Table 6-2 Concrete Classes	6-7
	Table 6-3 Concrete Testing Requirements	6-7
	Table 6-4 Maximum Deck Spans on Steel Girders	6-11
	Table 6-5 Standard Slab Designs	6-11
	Table 6-6 Maximum Deck Spans on Concrete Girders.....	6-13
	Table 6-7 Precast Deck Panels on Steel Girders.....	6-15
	Table 6-8 Concrete Unconfined Pullout Strength.....	6-16
	Table 6-9 Anchor Bolt Yield Strength (kips).....	6-16
	Table 6-10 Reinforcing Steel Yield Strength	6-16
	Table 6-11 Reinforcing Steel Anchorage	6-19
	Table 6-12 Additional Bar Length.....	6-19
	Figure 6-1 Concrete Slab on Steel Stringers	6-12
	Figure 6-2 Precast Deck Panels on Girder Superstructures	6-14

6 CONCRETE

6.1 Precast Concrete

6.1.1 General

6.1.1.1 Prestressing

To control stresses at the ends of prestressed beams, harping or debonding of strands may be specified. Harped strands allow for a more favorable distribution of stresses in the beam resulting in more efficient use of strands. Debonding does not allow the same degree of flexibility to control stresses throughout the beam. Every effort should be made to keep a straight strand pattern with a few debonded strands. However, in some cases, particularly with deep members such as New England Bulb Tees (NEBT), harping may be necessary. If harped strands are specified, the Structural Designer should be familiar with the practice and limitations of regional producers. For further guidance on this subject, consult with the Bridge Quality Assurance Team.

6.1.1.2 Camber, Deflection, and Blocking

The Structural Designer should consider camber, deflection, and blocking to avoid negative blocking in the structure. In the past, minimum practical haunch dimensions above the centerline of the top of the beam were found to be 2 inches for AASHTO I-girders and 3 inches for NEBT girders or spread box beams. These dimensions should be checked to determine their applicability to the design and increased if needed to avoid encroachment into the deck by the top flange.

Camber of the beams must be considered with the proposed centerline profile. Differences between the camber and the roadway profile can lead to a haunch or leveling slab that is too thin or one that is excessively thick. Methods for limiting or increasing camber can be found in PCI Precast Prestressed Bridge Design Manual (1997).

For staged construction projects and projects where beams may be fabricated more than four months prior to placement, the Structural Designer must consider the effects of camber growth.

Dead load deflections and a table of bottom of slab elevations should be given on the contract drawings for spread box, NEBT, and AASHTO I-girder structures. The Contractor should use screed rails to construct the finish

grade, as noted in Appendix D Standard Notes Precast Concrete Superstructures.

6.1.1.3 Section Properties

Section properties should be based on the concrete section alone, neglecting any effect of mild reinforcing steel or prestressing strand.

When designing precast beams composite with a concrete deck, section properties should be computed assuming a haunch dimension of one inch and an equivalent transformed width of deck. Dead loads should be based on the actual concrete dimensions.

6.1.1.4 Constructibility Check

Special consideration should be given to construction loading and the deck casting sequence. This sequence, including placing of the diaphragms, can directly affect the capacity of the superstructure components. The deck casting sequence should be specified on the plans, and should include instructions for placing diaphragms.

6.1.1.5 Allowable Stresses at Service Loads

When precast prestressed superstructures are used over salt water or other corrosive environments, under no circumstances should the tension at load combination Service III exceed the requirements of AASHTO LRFD Section 5.9.4.2.2, for the severe exposure condition.

6.1.2 *Materials*

6.1.2.1 Concrete

In general, precast concrete designs should be based on a 28 day compressive strength up to 6.5 ksi. Concrete strengths in excess of this should be used only when approved by the Engineer of Design, and when verified that regional precasters are capable of producing quality concrete at higher strengths. Precast concrete should be specified as Class P on the plans. The maximum permeability for the precast concrete should be indicated on the plans, which is 3000 coulombs in most cases.

Prestressed concrete units should contain a calcium nitrate corrosion inhibitor admixture, commonly referred to as DCI, in the concrete mix at a rate of 3 gal/yd³. This requirement is specified in Standard Specification Section 535 - Precast Prestressed Concrete Superstructure. For structures over salt water or other corrosive environments, the Structural Designer

should increase the rate of corrosion inhibitor to 5.5 gal/yd³. The Structural Designer must verify that the PS&E package contains a Special Provision for this requirement.

6.1.2.2 Prestressing Strand

Prestressing strand should be uncoated low relaxation seven wire strand meeting the requirements of AASHTO M 203 Grade 270. Strands for NEBT structures should typically be 1/2" diameter, with a maximum 0.6" diameter. The standard size of strands for prestressed beam slabs and boxes should typically be 0.6" diameter. Strands for precast deck panels should be a maximum 3/8 inch diameter, while all other strand should be a maximum diameter of 1/2".

Prestressing bars should be uncoated high strength steel bar meeting the requirements of AASHTO M 275.

6.1.2.3 Mild Reinforcement

Refer to Section 6.2.1.2 for reinforcement material requirements for non-prestressed reinforcement.

6.1.3 *Economy*

6.1.3.1 Release Strength

Concrete strength at release of prestress force can significantly affect cost. Precasters rely on daily use of their prestressing beds. Concrete strength at release is often the controlling factor in the concrete mix design. Excessive release strengths will either force the precaster to use higher strength concrete than the design requires or delay the release of prestressing force. The suggested release strength should be in the range of 4 to 4.5 ksi.

6.1.3.2 Beam Sections

When designing precast superstructures, uniform beam widths and strand patterns should be used whenever possible. Prestressing beds are long and can often accommodate more than one beam. Uniform beam widths and strand patterns allow more than one beam to be placed in the prestressing bed at a time, thus accelerating and economizing production.

6.1.4 Design Requirements

6.1.4.1 Concrete Cover

All precast main carrying members should be designed with the stirrups encasing all prestressing strands. The minimum cover for the stirrup is 1 inch from the bottom of the section.

6.1.4.2 Voided Slab and Butted Box Beam Bridges

A. Transverse Post-Tensioning

Normally, post-tensioning should be accomplished by the use of 0.6" diameter prestressing strand as specified in the applicable Supplemental Specifications. In cases where the chuck-to-chuck length is 25 feet or less, prestressing strand cannot be used due to excessive overstressing for the setting losses. For shorter post-tensioning lengths, the material and final tensile force must be clearly stated on the Plans. The tensile force should be 40k per location. The use of threaded rods such as DYWIDAG bars is recommended.

Commentary: The use of 0.6" diameter prestressing strand with a larger post-tensioning force is intended to limit cracking of the shear keys. Standard Detail 535(02) has been reviewed and approved for use with this larger strand size.

Diaphragms and strand locations should be spaced as described in Table 6-1. Diaphragms and post-tensioning ducts may be placed parallel to the centerline of bearing for skews less than 30°. For skews over 30°, diaphragms should be placed normal to the beams and consideration should be given to torsional loads from sidewalks, future widening, and maintenance of traffic. The end post-tensioning should be located such that it does not interfere with the wingwalls, including allowances made for the post-tensioning jack.

Table 6-1 Post Tension and Diaphragm Locations

Beam Type	Span	Ends	1/3 points	1/4 points and mid-span	Single mid-depth strand	Top and bottom strand
Voided Slabs	All	X	X		X	
Box Beams less than 3 ft deep	≤ 50 ft	X	X			X
	> 50 ft	X		X		X
Box Beams 3 ft and deeper	All	X		X		X

B. Wearing Surfaces

Refer to Section 4.7 Membranes for membrane requirements under pavement. Concrete wearing surfaces should be avoided unless a minimum 6 inch composite leveling slab is used.

C. Leveling slabs

In general, a reinforced composite slab should be used on all voided slab and butted box beam structures, with a minimum thickness of 4.5 inches at the curb line and a cross slope that matches the finished slope.

In some cases, an unreinforced leveling slab may be used, when approved by the Engineer of Design. The minimum thickness is 2 inches at the curb line, and the cross slope matches the finish slope. In rare cases, the concrete slab may be omitted based upon project specific considerations.

D. Continuity Design

Prestressed girders should be made continuous for the maximum practical length to avoid expansion joints. In general, the design should follow AASHTO LRFD Section 5.14.1.2 - Precast Beams. The Structural Designer is also referred to Oesterle (1989).

1. Negative Moment Over Piers

As a minimum, sufficient continuity steel should be provided to control cracking at the pier in the wearing surface at service loads. Crack control should be checked in AASHTO LRFD Section 5.7.3.4. The following values should be used for the crack width parameter Z:

Bituminous with high performance membrane	170 k/in
Concrete wearing surface*	77 k/in

*A crack width parameter up to $Z = 130$ k/in may be allowed with the use of galvanized or epoxy coated reinforcing steel and low permeability concrete.

Crack width parameters of 170 and 77 k/in correspond to approximate crack widths of 0.016" and 0.007" respectively. More refined methods of determining crack width such as the Gergely-Lutz equation for crack width are allowed.

2. Positive Moment Over Piers

As a minimum, sufficient continuity steel should create a reinforced section that resists 1.2 times the cracking moment.

|

E. Skew

Voided slab and butted box beam superstructures should not be used for bridges with skew angles greater than 45°. Bridges with heavy skews present problems with beam alignment during erection. Heavy skews also increase shear forces at the obtuse corners that may lead to shear key failure. Utilizing these beams with skews greater than 45° requires the approval of the Engineer of Design.

F. Transfer Length

MaineDOT utilizes long solid end diaphragms. The extended length eliminates spatial conflicts between the substructure and the end post-tensioning ducts. However, the solid end section sometimes extends to or beyond the transfer length section location. For such designs, some commercially available software will incorrectly apply the release prestress force to the much smaller voided cross section at the transfer length section location. The consequent error in the axial stress magnitude is much greater than the error in bending stress. Therefore, such software will undervalue the resultant top tensile stress.

When the solid end of a voided slab or box beam extends three inches or more beyond the transfer length section location, the Designer should manually analyze the top fiber tensile stress at the transfer length utilizing the solid section.

6.1.4.3 NEBT, AASHTO I-Girder, and Spread Box Beam Bridges

A. Diaphragms

Unless supported by integral abutments, end diaphragms should be designed to allow for jacking during future maintenance operations.

B. Continuity Design

Post-Tensioned Spliced NEBT Girder: The Structural Designer is referred to the PCI guidelines for post-tensioning and splicing NEBTs.

Conventionally Reinforced: The design should follow AASHTO LRFD. The Structural Designer is also referred to PCI (1997) as well as Oesterle (1989). Refer to Section 6.1.4.2D for further guidance.

C. Deck Overhang Limits

To control flexural stresses in the top flange of exterior beams, the overhanging portion of the CIP slab as measured from the edge of the top flange should be limited to 2 feet.

6.2 Cast-In-Place Concrete

6.2.1 Materials

6.2.1.1 Concrete

A. Concrete Class

There are four classes of concrete used for cast-in-place (CIP) structures: Class A, Class LP, Class S, and Class Fill. Guidelines on when to use each class are described in Table 6-2. Refer to Standard Specification Section 502 – Structural Concrete for further guidance.

Concrete strengths in excess of those described in the Standard Specifications should be used only when approved by the Engineer of Design, and when verified that regional suppliers are capable of producing quality concrete at higher strengths.

Table 6-2 Concrete Classes

Concrete Class	Concrete Use
Fill	Fill
LP	Structural Wearing Surfaces, Sidewalks, Curbs, Barriers, End Posts
S	Seals
A	All Others

B. Quality Control and Quality Assurance Guidelines

There are three possible methods for specifying the structural concrete for acceptable quality control (QC) and quality assurance (QA): Method A (Statistical Acceptance), Method B (Small Quantity Product Verification), or Method C (conforming to the requirements). Guidelines on the requirements of Method A, Method B, or Method C are described in Table 6-3. Incentives and disincentives are determined from the QA test results. Under Method A and B, if the test results indicate that the concrete quality is less than acceptable limits, the concrete may be removed and replaced at MaineDOT’s discretion.

Table 6-3 Concrete Testing Requirements

Method	QA Responsibility	QC Responsibility	Incentive	Disincentive
A	MaineDOT	Contractor	X	X
B	MaineDOT	Contractor		X
C	Contractor			

A Special Provision 502 must be included in all contracts that will designate under which method each concrete item will be classified (A, B, or C). The Structural Designer, together with the Construction Resident, must decide during the design phase of the project whether to specify Method A, Method B, or Method C concrete. If there is any

doubt, guidance from one of the Construction Engineers may be requested.

Guidelines on when to specify Method A, Method B, or Method C are as follows:

Method A should be specified where quality above the specification requirements is of value. Examples of where Method A is appropriate include, but are not limited to: footings, abutments, structural seals, piers, superstructures, decks, sidewalks, curbing, wearing surfaces, barrier, retaining walls, box culverts, bases for overhead sign supports, and mast arm traffic signal supports. P, the unit value for pay adjustment purposes, must be provided in the Special Provision that is included in each contract. P values reflect the price per cubic yard for all pay adjustment purposes. P values will be established on an annual basis and should not be based strictly on bid history information.

Method B should be specified where concrete must meet specifications but where there is no value added by quality exceeding the requirements of the specifications. Examples of where Method B is appropriate include, but are not limited to: approach slabs, concrete fill, pipe pile concrete, non-structural seals, traffic signal bases, and sign bases when not cantilevered. Method B may also be specified for the concrete items that normally call for Method A when the quantities are such that if Method A were specified, an inordinate amount of QA testing would be required and the benefit of specifying Method A over Method B would not differ significantly.

Method C should be specified where concrete quality still has to meet the specifications, but the benefits and costs to the Contractor and to the Department to develop and administer a Quality Control Plan, as required by specifications, are not justified. Examples of where Method C is appropriate include: armored joint repairs; surface repairs to wing walls, bridge decks, abutments, piers, and box culverts; and modifications to existing end-posts. This method should not be specified for structural elements that are expected to have a long design life.

6.2.1.2 Reinforcing Steel

Plain reinforcing steel should be deformed bars meeting the requirements of AASHTO M31 (ASTM A615). In general, the minimum bar size should be #5 for main reinforcing members and #4 for stirrups.

A corrosion resistant reinforcing system should be used for selected locations. The service life and cost of corrosion resistant systems can vary significantly. Some of these systems have not yet been used in Maine, so the Designer should use good engineering judgment in evaluating and

selecting appropriate options for each location. Corrosion resistant reinforcing systems include the following:

- A. MMFX reinforcing steel – a high strength, low carbon, chromium-rich deformed bar meeting the requirements of ASTM A1035. The use of this bar is currently limited to bridge decks, medians, curbs, sidewalks, barriers, and traffic rails. If a proposed design is utilizing the higher strength of MMFX, then there are several design issues that the Designer needs to consider. Ductility is of special concern. In designing the rebar layout, it is preferable to choose a smaller bar with similar spacing to a comparable standard strength design, as opposed to increasing the spacing of a similar sized bar. Too wide a spacing can result in increased cracking. This bar is a proprietary product and may only be used when the proprietary item approval process has been followed.
- B. Galvanized reinforcing steel - a deformed bar clad with zinc, meeting the requirements of ASTM A767. This bar must not be used with uncoated bars in the same structural element, as the galvanizing will sacrifice itself to protect the uncoated bars, thereby resulting in a reduced service life.
- C. Plain reinforcing steel used in conjunction with an approved corrosion inhibitor concrete additive. This is a suitable alternative for some applications, such as low volume and local roads that might not see as much salt intrusion.
- D. Stainless-clad reinforcing steel – a deformed bar clad with stainless steel, meeting the requirements of AASHTO MP13-04.
- E. Stainless reinforcing steel – a deformed bar made of solid stainless steel, meeting the requirements of ASTM A955.
- F. ZBAR reinforcing steel – a dual-coated deformed bar with a metallized zinc inner layer under a polymer outer layer, meeting the requirements of ASTM A1055. This bar is a proprietary product and may only be used when the proprietary item approval process has been followed.
- G. GFRP reinforcing bar – a glass fiber-reinforced polymer bar with a low modulus of elasticity, meeting the requirements of ACI 440.1R-06. The use of this bar is currently limited to bridge decks, medians, curbs, sidewalks, barriers, and traffic rails.
- H. Epoxy-coated reinforcing steel – a deformed bar clad with epoxy coating, meeting the requirements of AASHTO M31 (ASTM A615) and AASHTO M284 (ASTM A775). There are conflicting research reports regarding how effective this bar really is. It can be effective if handled properly and the coating remains completely intact; however, preventing damage to the epoxy coating in the field is

virtually impossible. Corrosion easily takes place in areas where the coating is damaged. Disbonding of the coating from the bar is a concern as well. Other corrosion resistant reinforcing systems are preferred over this one.

The following locations in concrete bridge elements should incorporate the use of a corrosion resistant reinforcing system:

A. Substructure

- All pier columns, shafts, and caps of grade separation structures that are within 30 feet of the traveled way, including footing dowels if they extend above the finished grade line
- All abutment bridge seats and front faces of breastwalls of grade separation structures that are within 20 feet of the traveled way, including footing dowels if they extend above the finished grade line
- The front face of all retaining walls and wingwalls of grade separation structures that are within 20 feet of the traveled way, including footing dowels if they extend above the finished grade line
- All substructure units in their entirety, when the bridge passes over salt water

B. Superstructure

- All curbs, sidewalks, medians, barriers, and endposts
- All deck slabs when the bridge passes over salt water
- All deck slabs of continuous steel structures with concrete wearing surfaces

Other locations, as approved by the Engineer of Design, may also incorporate a corrosion resistant reinforcing system where it is considered to be cost effective. In addition, the Engineer of Design may approve the elimination of a corrosion resistant reinforcing system at locations where it may not be cost effective, due to low traffic volumes and/or low susceptibility to salt intrusion.

6.2.2 Decks

The deck slab should be carried over the abutment backwall under the following circumstances:

CHAPTER 6 - CONCRETE

- o On abutments with fixed bearings when beam depth is less than approximately 4 feet. On roads with low traffic volume, the Structural Designer may choose to carry the slab over the backwall for beams deeper than 4 feet.
- o On abutments with expansion bearings for bridges within the following limits:
 1. Spans up to 40 feet with skew up to 45°

2. Spans up to 50 feet with skew up to 40°
3. Spans up to 60 feet with skew up to 35°
4. Spans up to 70 feet with skew up to 30°
5. Spans up to 80 feet with skew up to 20°

Concrete curbs should be placed continuously with extra longitudinal steel in the top of the curb over piers. This additional reinforcement should extend into the positive moment region not less than the development length of the bar. Sidewalks on bridges are treated in the same manner.

When a deck slab on new girders will be built in successive stages, or when staged construction is used to replace an existing slab, a zipper strip should be considered where sufficient width is available to maintain traffic. A zipper strip is a longitudinal concrete closure pour between two successive deck construction stages. A zipper strip is intended to reduce the effect of adjacent live loads during curing and to minimize cracking between stages.

Based upon the anticipated use of completed parts of a structural slab during construction, the Structural Designer may wish to specify that the formwork be designed to carry all or part of the design live load.

When designing the superstructure slab for a multiple span continuous structure with more than 250 yd³ of deck concrete, an optional deck construction joint must be provided for use if the Contractor elects to place the deck concrete in successive placements. Refer to Appendix D Standard Notes Superstructures.

6.2.2.1 Standard Design of Concrete Slab on Steel Girders

Table 6-4 shows the maximum span for a given slab thickness on steel girders and Table 6-5 shows standard reinforcing steel design for concrete slabs in superstructure slab designs. For these tables, a 3 inch bituminous wearing surface with 1/4" membrane was used in the slab designs. The slab design should not be modified for a lighter weight wearing surface. Refer to Figure 6-1 for an explanation of reinforcing details.

This design uses straight bars top and bottom, without the use of crank bars. If precast deck panels will not be offered as an option to the Contractor (refer to Section 6.2.2.3), the Structural Designer may choose to specify crank bars instead.

Extra distribution bars in negative moment areas should be designed in accordance with AASHTO LRFD Section 6.10.3.7 - Minimum Negative Flexure Slab Reinforcement. In general, the requirement will be met if the

bottom mat of distribution steel is as shown in the standard design in Figure 6-1 and the top mat of distribution steel is changed to accommodate the increased requirement for steel. If possible, the top mat of steel should have only one size bar with a minimum spacing of 6 inches.

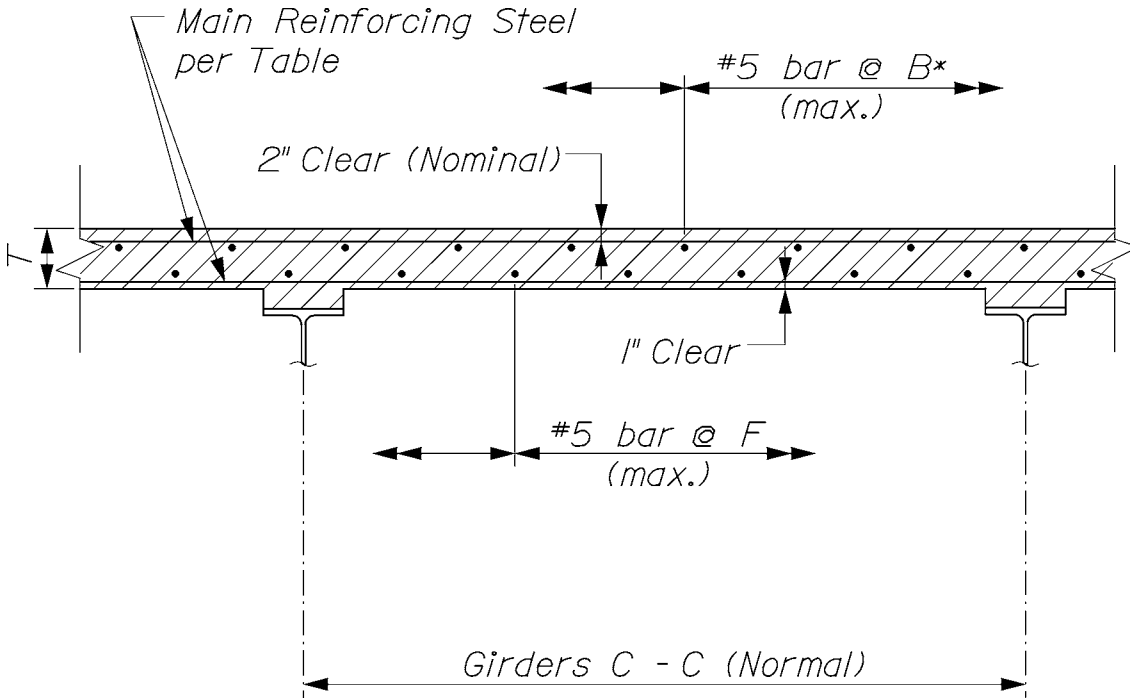
Table 6-4 Maximum Deck Spans on Steel Girders

Slab Thickness (in)	Maximum Girder Spacing		
	Skew (θ)		
	$0^\circ < \theta \leq 10^\circ$	$10^\circ < \theta \leq 25^\circ$	$\theta = 0^\circ, \theta > 25^\circ$
7	7'-1"	8'-4"	7'-3"
7 ½	8'-7"	8'-10"	8'-9"
8	9'-4"	9'-6"	9'-6"
8 ½	9'-10"	9'-11"	10'-0"
9	10'-4"	10'-5"	10'-6"
9 ½	10'-10"	11'-1"	11'-0"
10	11'-1"	11'-6"	11'-3"
10 ½	11'-7"	12'-2"	11'-9"
11	12'-0"	12'-10"	12'-3"

Table 6-5 Standard Slab Designs

Slab T (in)	Main Reinforcing			Dim F (in)	Dim B (in)
	Skew (θ)				
	$0^\circ < \theta \leq 10^\circ$	$10^\circ < \theta \leq 25^\circ$	$\theta = 0^\circ, \theta > 25^\circ$		
7	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	7	18
7 ½	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	7	18
8	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	7	18
8 ½	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	7	18
9	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	7	18
9 ½	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	7	15
10	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	7	15
10 ½	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	8	15
11	#5 @ 6"	#5 and #6 alternating @ 6"	#5 @ 6"	8	15

Note: The spacing for the main reinforcing steel is measured along the centerline of beam. The main reinforcing steel is parallel to the skew for skews less than or equal to 25° and perpendicular to the girders for skews greater than 25° .



TRANSVERSE SECTION

* For extra distribution steel in negative moment areas, see text.

DESIGN: Load & Resistance Factor Design per AASHTO LRFD - Specifications for Highway Bridges 1998 and Interim Specifications through 2002

LOADING: HL-93 Modified for Strength I (Impact = 33%)

STRESSES: Reinforcing steel ~ $f_y = 60,000$ psi
 Concrete ~ $f'_c = 4,350$ psi (Cast - in - place)

Figure 6-1 Concrete Slab on Steel Stringers

6.2.2.2 Standard Design of Concrete Slab on Concrete Girders

Table 6-6 shows the maximum span for a given slab thickness on concrete girders and Table 6-5 shows standard reinforcing steel design for concrete slabs in superstructure slab designs. This table assumes a 3 inch bituminous wearing surface with 1/4" membrane, and a minimum top flange width of 3 feet. The slab design should not be modified for a lighter weight

wearing surface. Refer to Figure 6-1 for an explanation of reinforcing details.

Table 6-6 Maximum Deck Spans on Concrete Girders

Slab Thickness (in)	Maximum Girder Spacing		
	Skew (θ)		
	$0^\circ < \theta \leq 10^\circ$	$10^\circ < \theta \leq 25^\circ$	$\theta = 0^\circ, \theta > 25^\circ$
7	8'-10"	9'-9"	9'-0"
7 ½	9'-10"	10'-2"	10'-0"
8	10'-7"	10'-8"	10'-9"
8 ½	11'-1"	11'-4"	11'-3"
9	11'-6"	12'-0"	11'-9"
9 ½	12'-0"	12'-5"	12'-3"

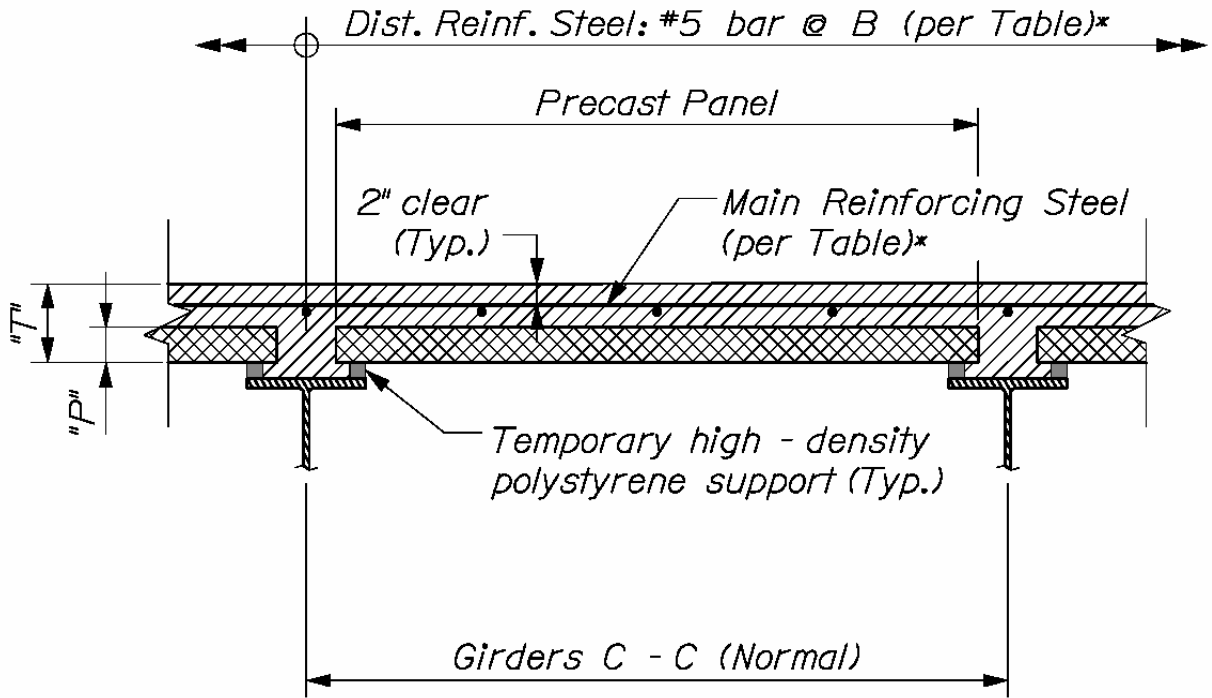
6.2.2.3 Precast Deck Panels

The Contractor may be given the option of constructing the concrete deck with precast, prestressed concrete deck panels. Standard designs of precast deck panels on steel girders are covered in this section. For structures with wider flanges and smaller girder spacings, precast deck panels should incorporate plain reinforcement along with prestressed reinforcement.

Precast, prestressed concrete deck panels on steel girders are specified in accordance with Table 6-7. Refer to Figure 6-2 for an explanation of reinforcing details. The panel type and number of reinforcing strands should be indicated on the design drawings.

Standard Details 502 (07-12) do not include details for systems with slabs thinner than 8 inches. The precast deck panel cannot be used with toppings thinner than 4-3/8" because there is insufficient cover for the topping reinforcing steel. Therefore, there is no direct precast system substitution for the standard slab designs with 7" and 7-1/2" slabs. Eight inch slabs cannot be substituted due to difficulties with profile grades/ bottom of slab grades. However, designs for the 7'-0", 7'-6", 8'-0", and 8'-6" design span slabs are included in Table 6-7 so that the Structural Designer can use these shorter spans for projects that specify solely precast, prestressed panels. Spans shorter than 7 feet cannot be designed due to the inability to develop the strength of the prestressing strand.

Precast, prestressed deck panels should not be used when skews exceed 30°.



TRANSVERSE SECTION

* For extra distribution steel in negative moment areas, see Section 6.2.2.1

DESIGN: Load & Resistance Factor Design per AASHTO LRFD- Specifications for Highway Bridges 1998 and Interim Specifications through 2002

LOADING: HL-93 Modified for Strength I (Impact = 33%)

*STRESSES: Prestressing steel ~ $f's = 270,000$ psi
 $E_s = 78,500,000$ psi
 Reinforcing steel ~ $f_y = 60,000$ psi
 Concrete ~ $f'c = 4,350$ psi (Cast - in - place)
 $f'c = 6,000$ psi (Precast)*

MATERIALS: Strands ~ $\frac{3}{8}$ " ϕ , Grade 270 (Low relaxation)

Figure 6-2 Precast Deck Panels on Girder Superstructures

The deck panels and the main reinforcement should be normal to the girders. Superelevated and curved bridges require special consideration for variable temporary blocking thickness and non-parallel panel layouts.

Table 6-7 Precast Deck Panels on Steel Girders

Panel Type	Maximum Girder Spacing	Slab "T" (in)	Panel "P" (in)	Number of Strands		
				Compression Flange Width		
				1'	1'-6"	2'
A1	7'-6"	8.0	3.5	15	15	15
A2	8'-0"	8.0	3.5	15	15	15
A3	8'-6"	8.0	3.5	17	16	16
A4	9'-0"	8.0	3.5	19	17	17
A	9'-6"	8.0	3.5	21	19	18
B	10'-0"	8.5	3.5	22	21	19
C	10'-6"	9.0	3.5	24	22	20
D	11'-0"	9.5	3.5	27	24	22
E	11'-6"	10.0	3.5	30	27	25
F	12'-0"	10.5	3.5	33	30	28

The Structural Designer should indicate on the design drawings the size and location of reinforcing steel for the top mat and for the cast-in-place end sections as indicated in Section A - A of Standard Detail 502 (08) Precast Concrete Deck Panels.

The appropriate notes found in Appendix D Standard Notes Superstructures should be included in the contract drawings.

6.2.3 Drilled and Anchored Bolts/Bars

There are two general conditions where drilled and anchored bolts or reinforcing steel will be used. The first is where adequate concrete thickness is available to develop the yield strength of the anchor. The second is where adequate concrete thickness is not available to develop the yield strength of the anchor.

When adequate concrete thickness is available, anchorage will be designed for the yield strength of the anchor. The unconfined pullout strength specified on the plans will equal the yield strength of the anchor (refer to Table 6-9 and Table 6-10).

When adequate concrete thickness is not available, the design capacity of the anchor will be limited by the unconfined pullout strength of the concrete (refer to Table 6-8).

The following tables may be used for the design of drilled and anchored bolts and reinforcing bars in lieu of a more precise analysis. The strengths given are ultimate strengths and therefore appropriate load factors should be applied to design loads. A concrete compressive strength of 3 ksi is assumed.

Table 6-8 Concrete Unconfined Pullout Strength

Depth (in.)	3	4	5	6	7	8	9	10	12	15	18	24
Unconfined Pullout (kips)	3.5	7	11	17	24	32	40	50	70	120	170	300

Table 6-9 Anchor Bolt Yield Strength (kips)

Bolt Size (in.)	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\text{-}\frac{1}{8}$	$1\text{-}\frac{1}{4}$	$1\text{-}\frac{3}{8}$	$1\text{-}\frac{1}{2}$
A449 & A325	6.5	12	19	28	39	51	56	71	85	103
A709 Grade 50	3.5	7	11	16	23	30	38	48	57	70
A709 Grade 36	2.5	5	8	12	16	21	27	34	41	50

Table 6-10 Reinforcing Steel Yield Strength

Bar Size (#)	3	4	5	6	7	8	9	10	11	14	18
Yield (kips)	6.5	12	18	26	36	47	60	76	93	135	240

6.2.3.1 Specification Procedures

The following information should be provided on the plans when specifying drilling and anchoring.

- Anchor size
- Anchor spacing or layout
- Anchor type
- Unconfined pullout requirements
- Minimum anchor embedment depth

Anchors set by drilling and anchoring have been divided into three general types:

- Type I - Anchor bolts size one inch or greater
- Type II - Anchor bolts smaller than one inch
- Type III - Reinforcing steel anchors

A list of prequalified anchoring materials for each type of anchor is available at <http://www.maine.gov/mdot/utilities/uap.php>. Appropriate notes from Appendix D Standard Notes Drilled and Anchored Bolts and Reinforcing Steel should be included on the plans.

The minimum embedment depth given on the plans is based on the depth required to achieve adequate concrete strength. Additional depth above Table 6-8 requirements may be specified, if the Structural Designer feels it is required, as the added cost of increased embedment depth is minimal. However, the embedment should not be less than shown in Table 6-8 without a more precise analysis or a proof load test.

When available concrete thickness is not adequate to provide unconfined pullout strength equal to the yield of the anchor, or the condition of the concrete is a concern, a proof load test may be specified. This can be done by including Supplemental Specification, Section 502 (Proof Load Testing) in the contract book and including the appropriate pay items.

Because of limitations of readily available testing equipment, proof load tests should not be specified for unconfined pullouts in excess of 50 kips. If an unconfined pullout test greater than 50 kips is needed, the Structural Designer should consult with MaineDOT's Transportation Research Division to determine the availability and practicability of specifying a proof load test.

A. Type I Anchors

Bearing plate anchor bolts sizes 1" and 1-1/2" are specified in the Standard Details. For other sizes of bearing anchor bolts, specify the minimum embedment depth and anchor bolt size.

For all other anchor bolts, specify the anchor bolt as a Type I anchor and include the appropriate notes found in Appendix D Standard Notes. Specify the bolt size, spacing, minimum embedment depth (from Table 6-8), and the unconfined pullout requirements.

B. Type II Anchors

For bridge rail anchors mounted on curbs with adequate concrete depth, include Special Provision Section 507 - Railings (Anchor Bolt Installation). When available embedment is less than is required in Special Provision 507, the Structural Designer should either do a more precise analysis or use other methods of attaching anchor bolts.

For all other anchor bolts, specify that the anchor is a Type II anchor and include the appropriate notes from Appendix D. Specify the bolt size, spacing, minimum embedment depth (from Table 6-8), and unconfined pullout requirements.

C. Type III Anchors

When using drilled and anchored reinforcing bars, specify that they are Type III anchors and include the appropriate notes from Appendix D. Specify rebar size, spacing, minimum embedment depth (from Table 6-8), and unconfined pullout requirements.

For concrete curb and barrier rail reinforcing steel anchors, use Table 6-11 when appropriate.

Add additional bar length to the dimensions in the reinforcing steel schedule (for embedment) according to Table 6-12, or to the maximum available embedment if less. The added bar length is to account for the fact that some products on the approved list may require embedment length greater than the minimum given on the plans, which is based on concrete strength only.

Added bar lengths must be equal to or greater than the embedment depth actually specified on the plans.

Table 6-11 Reinforcing Steel Anchorage

Curbs with Steel Bridge Railing			
Minimum Available Embedment Depth (in)	Minimum Bar Size	Maximum Bar Spacing (in)	Unconfined Pullout (k)
4	# 5	6	4
4-1/2	# 5	9	6
5	# 5	12	8
6	# 5	18	12
F-Shaped Concrete Barrier			
5	# 5	6	8
5-1/2	# 5	9	12
6	# 5	12	16
7-1/2	# 6	18	24

Notes:

1. Minimum available embedment depth is defined as slab thickness minus 2".
2. Curb requirements for steel bridge rail assume a 1'-5" minimum concrete curb width.
3. Concrete barrier requirements assume either 32" or 42" barrier height.

Table 6-12 Additional Bar Length

Bar Size	Bar Length
#4 & #5	12"
#6 & #7	15"
#8 & #9	18"

References

AASHTO, 1998 and Interims, *Load and Resistance Factor Design (LRFD) Bridge Design Specifications*, Washington, DC

Oesterle, R., 1989, Design of Precast Prestressed Bridge Girders Made Continuous, *NCHRP Report 322*

Precast Concrete Institute, 1997, *PCI Precast Prestressed Concrete Bridge Design Manual*, USA

Precast Concrete Institute New England Region Technical Committee, *Design Guidelines for Post-Tensioning and Splicing the NEBT Girder*

Chapter 7

STEEL



Turner Bridge, Turner



Kezar Falls Bridge, Parsonsfield-Porter

CHAPTER 7 - STEEL

7	STEEL	7-1
7.1	General.....	7-1
7.1.1	Blocking and Camber	7-1
7.1.1.1	Bottom-of-Slab Elevations	7-1
7.1.1.2	Blocking Detail.....	7-1
7.1.1.3	Camber Diagram	7-3
7.1.2	Section Properties	7-6
7.1.3	Constructability	7-6
7.2	Materials.....	7-6
7.2.1	Structural Steel.....	7-6
7.2.2	Higher Strength Bridge Steel.....	7-7
7.2.3	Coatings	7-7
7.2.3.1	New Steel.....	7-7
7.2.3.2	Existing Steel.....	7-7
7.2.3.3	Galvanizing	7-8
7.2.4	Availability	7-8
7.2.5	Bolts, Nuts, and Washers	7-8
7.2.6	Welds	7-8
7.2.7	Field Splices	7-9
7.3	Economy.....	7-9
7.4	Design Requirements	7-10
7.4.1	Welded Girders	7-10
7.4.2	Rolled Beams	7-10
7.4.3	Fabrication Considerations.....	7-11
7.4.4	Diaphragms and Cross Frames.....	7-12
7.4.4.1	Instructions for Use of Standard Details.....	7-12
7.4.5	Stiffeners and Diaphragm Connection Plates.....	7-13
7.4.5.1	General	7-13
7.4.5.2	Effect of Skew	7-14
7.4.6	Handhold Bars.....	7-15
7.4.7	Slab Overhang Limits	7-18
7.4.8	Composite Design	7-18
	References	7-19
	Table 7-1 Steel Fabricators.....	7-8
	Table 7-2 End Diaphragms	7-12
	Table 7-3 Intermediate Cross Frames	7-13
	Table 7-4 Slab Overhang Limits.....	7-18
	Figure 7-1 Theoretical Blocking Details	7-3
	Figure 7-2 Camber Details.....	7-5
	Figure 7-3 Stiffener/Connections Plate Details A Through C	7-16
	Figure 7-4 Stiffener/Connections Plate Details D and E	7-17
	Example 7-1 Maximum Cover Plate Width	7-11

7 STEEL

7.1 General

7.1.1 *Blocking and Camber*

7.1.1.1 Bottom-of-Slab Elevations

A Blocking Table containing bottom-of-slab elevations should be provided on the design plans for bridges having concrete slabs on steel girders. Bottom-of-slab elevations are used to set formwork after steel erection. No adjustment of beam dead loads is needed to calculate these elevations. For the calculation, the fluid and superimposed dead load deflections are added to the in-service bottom-of-slab elevations. These results are shown in the Blocking Table on the plans.

The Contractor will use the bottom-of-slab elevations in the Blocking Table to set the deck forms relative to the steel at each blocking point (bottom-of-slab elevation point), which will result in the correct profile of the constructed deck after deflection has occurred. The bottom-of-slab elevations are tabulated at regular intervals (generally 10 feet) for each span, beginning at the centerline of bearings. The last space in a span may be 10 feet or less.

7.1.1.2 Blocking Detail

The appropriate note from Appendix D Standard Notes Structural Steel should document the theoretical blocking used at each support to establish the position of the slab relative to the steel girders. The theoretical blocking is defined as the theoretical distance between the bottom of slab and the top of web for welded girders, and as the theoretical distance between the bottom of slab and top of flange for rolled beams, disregarding any cover plates. This definition avoids confusion caused by the abrupt changes in measured blocking thicknesses at cover plates and changes in plate girder top flange thickness. Refer to Figure 7-1.

It is necessary to use blocking because the profile of the steel will vary from the theoretical profile. Without the blocking, the steel could encroach into the bottom of the slab, or the slab profile would have to deviate from the design profile. The theoretical blocking on new construction should be established to provide an actual minimum of 1-1/2" clear distance between the bottom of slab and the top of steel point. The top of steel point is defined as top of flange on welded shapes and top of any cover plate on rolled shapes, but does not include any splice plates. Included in this 1-1/2" is 1 inch used in design to compute section properties of composite

beams or girders, plus an additional 1/2" to compensate for construction tolerances. In a connection with a splice plate thickness greater than 1 inch, the reference ordinate to the splice may be reduced to insure that the splice plate will not protrude into the slab. This is addressed again in Section 7.1.1.3 Camber Diagram.

In cases where span lengths exceed 100 feet, the clear distance between the reference point and the bottom of slab should be increased from 1-1/2" to 2-1/2" to further compensate for allowable fabrication tolerances. This clear distance should be increased even more, up to 3" or 3-1/2" when continuous spans with multiple placements are likely, to compensate for construction tolerances. However, this large clear distance may result in excessive blocking of greater than 1-1/2" at supports of long span continuous bridges. Another alternative to increasing the clear distance to this level is to arbitrarily flatten the camber of long girders by up to 1 inch, by detailing the camber diagram flatter than predicted by dead load deflections (refer to Section 7.1.1.3 Camber Diagram). This will minimize excessive blocking at supports. This will also increase the mid-span blocking measured in the field, will reduce the projection of shear studs into the deck, and may result in a sag in the steel on bridges not designed on a crest curve. These limitations should be carefully analyzed by the Structural Designer to determine the best fit blocking and camber for each bridge.

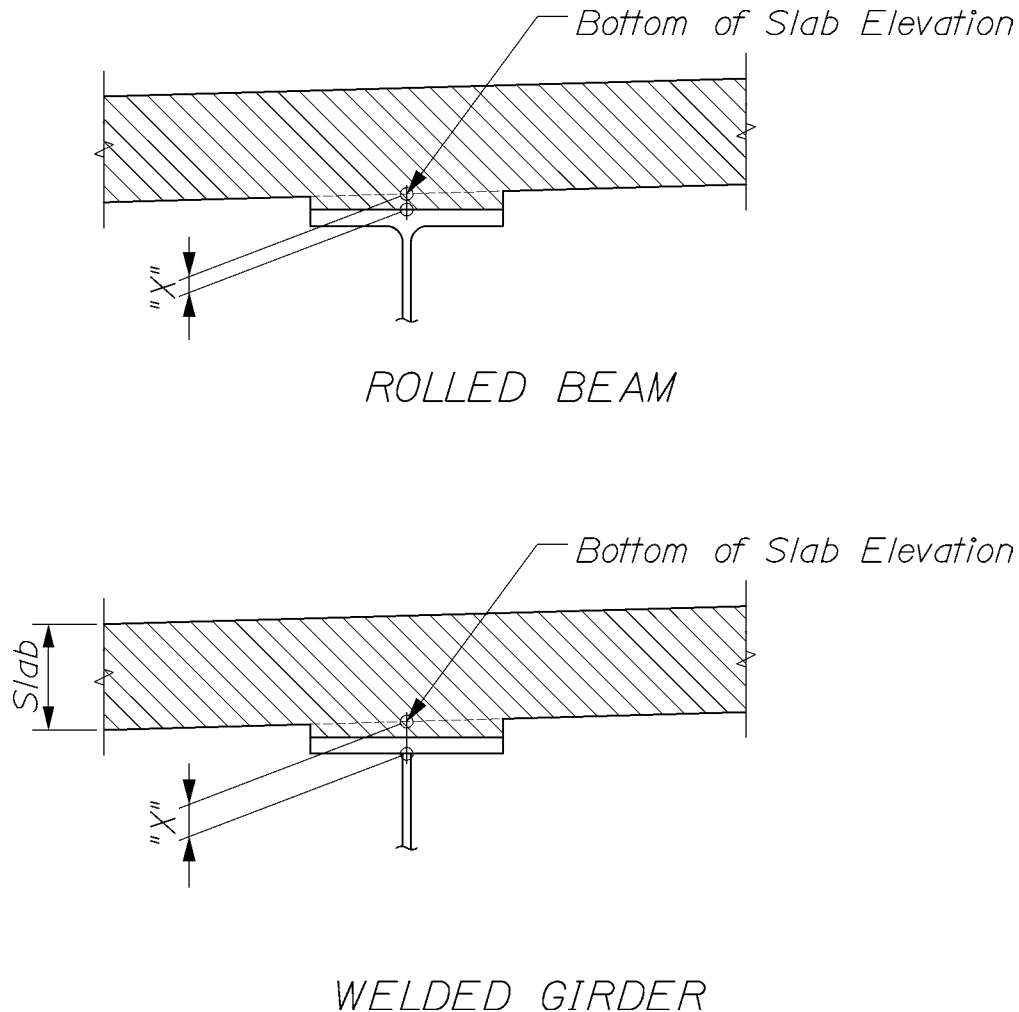


Figure 7-1 Theoretical Blocking Details

7.1.1.3 Camber Diagram

A camber diagram should be provided on the plans for all steel bridges except for simple span rolled beams. Camber is utilized to compensate for steel, fluid, and superimposed dead load deflections in order to attain the finished grades shown on the plans, and to avoid a sag in the steel or excessive blocking at mid-span. In the case of continuous span steel bridges, reference ordinates are shown from a level reference line to the beam at each support and field splice. Refer to Figure 7-2 for guidance. In accordance with design assumptions, the ordinates to the field splices are computed for the beam as if it were fabricated and erected in a weightless state. Some fabricators work with the steel lying on its side (weightless), and other fabricators work with the steel in a vertical position, resulting in some deflection from the weight of the steel. The former method is more in conformance with design assumptions, but both methods seem to work

adequately. Reference ordinates to splice points may need to be adjusted as discussed in Section 7.1.1.2 Blocking Detail to insure that the splice plates have an actual clearance to the bottom of slab of at least 1 inch.

Camber ordinates for welded girders, as well as rolled beams that will be heat cambered, are computed for an equal number of spaces from support to support for simple spans, and from the centerline of support to the centerline of splice for continuous spans. The spaces should be about 10 feet. Only an ordinate at the mid-length point should be specified for rolled beams that will be cold cambered; no other ordinates are needed or specified. When specifying camber for rolled beams, use the "Limits and Tolerances for Mill Cambering" as outlined in the AISC Manual of Steel Construction to the greatest extent as possible.

If the camber required for design is greater than can be achieved with cold cambering, then a combination of cold cambering and heat cambering, or heat cambering alone can be used. Camber should not be specified between 0 inches and the lower cold cambering limits.

In any case, Natural Mill Camber should not be used as an actual reliable quantity to give the camber required by design. The limits of camber, shown in the AISC Manual of Steel Construction and in ASTM Standard A6/A6M, are straightness tolerances for acceptance of as-rolled material only. All rolled beams are straightened at the mill after rolling. However, if the desired camber is between 0 inches and the lower limits for cold cambering, the Structural Designer should provide a blocking arrangement that can tolerate 0 inches of camber, and then specify Natural Mill Camber (either up or down as necessary) on the plans.

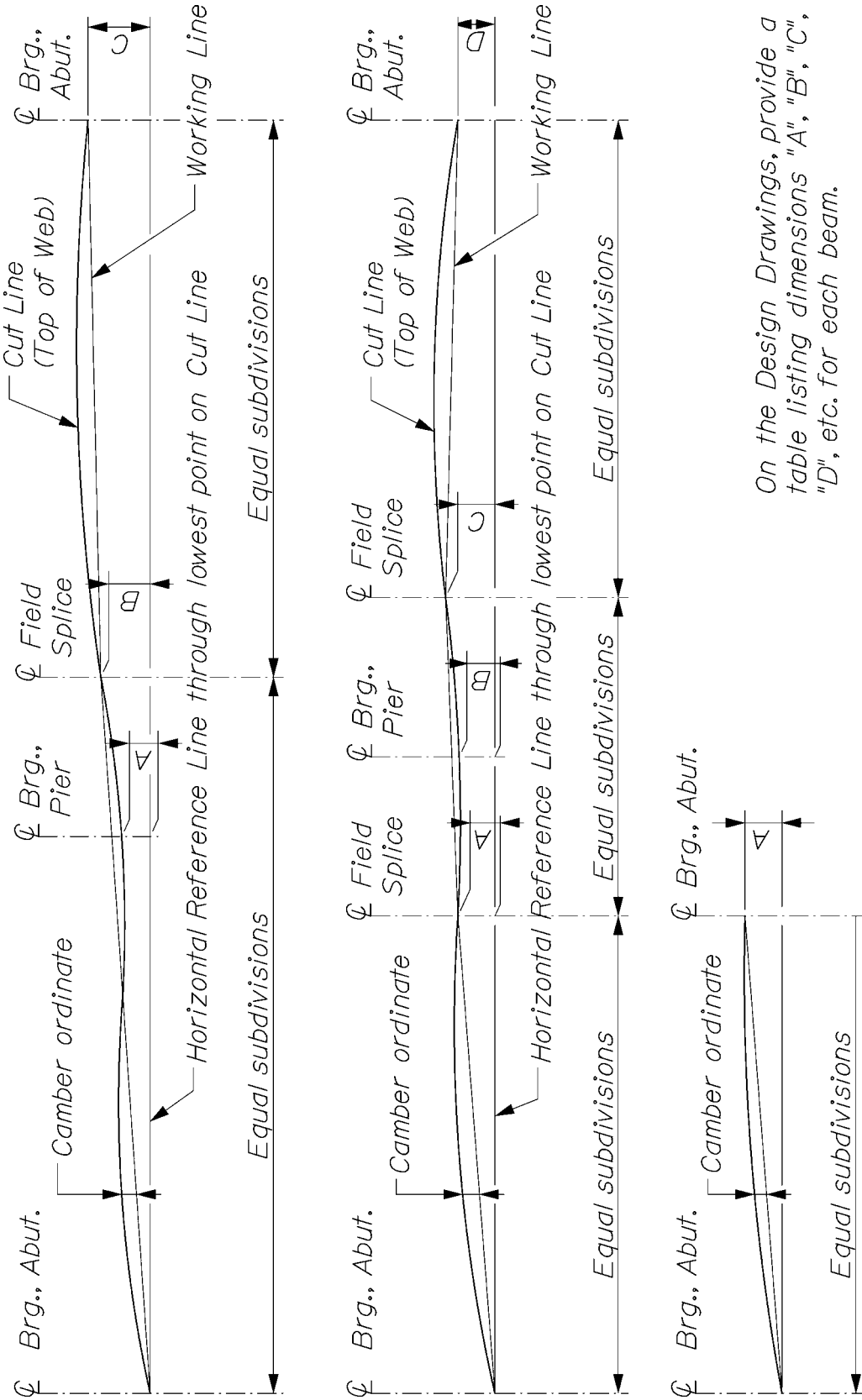


Figure 7-2 Camber Details

On the Design Drawings, provide a table listing dimensions "A", "B", "C", "D", etc. for each beam.

7.1.2 Section Properties

When designing beams with composite concrete decks, composite section properties should be computed assuming a haunch dimension of 1 inch and an equivalent transformed width of deck.

7.1.3 Constructability

Structural Designers should be familiar with constructability issues, and incorporate good practices in their designs. An excellent resource is the AASHTO/NSBA website at <http://www.steelbridge.org/>.

7.2 Materials

7.2.1 Structural Steel

Unpainted ASTM A709 Grade 50W steel (weathering steel) should be used for structures over water, except when such structures have open roadway joints or are located in a coastal, salt spray, or heavy industrial area. Unpainted ASTM A709 Grade 50W steel may be used for structures over railroads and highways except for narrow depressed roadways and similar situations that create tunnel-like conditions.

Weathering steel is resistant to only certain types of atmospheric corrosion. Weathering steel will not develop a protective oxide coating if it remains wet more than 60% of the time. Also, an excessive amount of contaminants in the air or the presence of salt will prevent the oxide coating from forming. For more information on this subject, refer to FHWA Technical Advisory (1989).

Painted, metallized, or galvanized ASTM A709 Grade 50 steel may be used where weathering steel is inappropriate, but only if a concrete superstructure is not a feasible alternative. Refer to Section 7.2.3 for coating requirements.

H-Piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel. Pipe piles used for bridge foundations should conform to the requirements of ASTM A252 Grade 2

Grade 3 with either straight or spiral butt-welded seams. Lap welded seams are not allowed.

7.2.2 *Higher Strength Bridge Steel*

This section will be written in the future.

7.2.3 *Coatings*

7.2.3.1 New Steel

In areas where the basic design criteria restricts the use of unpainted ASTM A709 Grade 50W steel, or in cases where a painted steel system is desired, a shop-applied, three-coat, zinc-rich coating system should be used with some field touch-up to repair any erection damage. The MaineDOT Standard Specifications do not address painting of structural steel; therefore, a Supplemental Specification needs to be provided in the PS&E package when a painted steel system is to be used.

If a painted steel system is desired, the Structural Designer should specify Type 1 bolts galvanized in accordance with ASTM A153. When unpainted weathering steel is used, only Type 3 bolts should be used, which are always plain.

The Contractor must select a coating system from the Northeast Protective Coating Committee (NEPCOAT) Qualified Products List (QPL). This list may be found through MaineDOT's QPL website: <http://www.maine.gov/mdot/transportation-research/approved-products.php>. The Structural Designer should consult with the coatings technical resource personnel to discuss the appropriate use of the specification.

7.2.3.2 Existing Steel

When developing a field paint project, the Structural Designer must bear in mind certain environmental and safety considerations that will require the containment of the blast medium used to remove the existing coatings and blasted material. These situations may result in a decrease in underclearance, requiring that provisions for maintenance of traffic and/or sequencing of operations be described in a Special Provision. Existing utility companies should be contacted through the Utility Coordinator to determine if there is a need for protecting any utility during construction. As with new steel, a NEPCOAT pre-qualified system must be used.

7.2.3.3 Galvanizing

Galvanizing is typically used on small plates, bridge railings, joint armor, and accessories. A top coat may be specified for aesthetic considerations. Galvanizing main beams may be considered for relatively short span rolled beams, but only if weathering steel is not an option. Approval from the Engineer of Design must be obtained for galvanizing main beams.

7.2.4 Availability

There are a limited number of steel suppliers of various shapes that satisfy the Buy America requirement included in the majority of MaineDOT contracts. This may lead to issues relating to excessive lead time for particular components.

Only one mill (Nucor-Yamato) in the United States produces all W40 shapes and the heavier W36 and W30 shapes. This mill and one other (TXI Chaparral) produce the remaining W36, W33, and W30 shapes. Lead time for W shapes is from 12 to 14 weeks.

Currently, these two mills also supply all HP shapes in the United States. Lead time for these shapes is from 8 to 12 weeks.

Through the information provided in Table 7-1, the Structural Designer should confirm the required lead time for the designed shapes. This will affect the planned construction schedule and necessary advertising date.

Table 7-1 Steel Fabricators

Company	Location	Website
Nucor-Yamato	Blytheville, AR	www.nucoryamato.com
TXI Chaparral	Midlothian, TX	www.chaparralsteel.com/

7.2.5 Bolts, Nuts, and Washers

Bolted field splices and other structural applications should use 7/8" diameter ASTM A325 High Strength Bolts. ASTM A490 bolts should not be used unless approved by the Engineer of Design.

7.2.6 Welds

Welds should be designed in accordance with the applicable AASHTO LRFD standard. Welds for cover plates, plate girders, and bearing stiffeners should be shown on the plans.

7.2.7 *Field Splices*

Bolted field splices should be designed as slip-critical. Uncoated weathering steel should be designed for Class B (slip coefficient 0.55) faying surfaces. For painted surfaces, refer to the approved coating list for the appropriate slip coefficient. The Structural Designer should not indicate the thickness of filler plates for splices on the plans. Allowable construction tolerances may affect these thicknesses, which are easily adjusted by the fabricator.

The splice design should provide adequate clearance to apply an impact wrench to any of the fasteners in the web or flanges; in other words, the extreme rows of bolts in the web should have a clear distance from the flange bolt assemblies.

AASHTO and NSBA documents provide minimum bolt hole to edge distances in splice plates and associated components. It is an advantageous design to provide a distance of 1-3/4 inches from the center of a bolt hole to a plate edge and a distance of 4 inches between rows of bolts straddling the girder field splice. This provides fabricators with a tolerance that is manageable for minimal extra cost and is within code guidelines for a 7/8" diameter bolt.

7.3 **Economy**

The Structural Designer should keep in mind that a design utilizing the least material is not necessarily the most economical design, since material cost represents only about one third of the total fabricated cost of a welded girder. The bulk of the cost lies in fabrication, shop fit-up, delivery, and field erection. Simplification and repetition of details, reduction of fabrication and welding operations, and ease of handling and erection are often better means to achieve cost savings.

As a general rule, unstiffened webs should be used for depths of 50 inches and below. For web depths over 50 inches, unstiffened or partially stiffened webs should be used. To determine an optimum number of intermediate stiffeners for a partially stiffened web, a cost of \$150 to \$200 per stiffener can be assumed.

At least 800 pounds of flange material must be saved to justify the introduction of a shop flange splice. Normally, the most economical design results when the flange sizes are carried through the entire positive moment section. It may or may not be cost effective to transition flange sizes in the negative moment section. If a flange transition is specified, the thickness and not the width should be varied, since a uniform flange width allows welding of an entire slab of steel rather than individual pieces.

CHAPTER 7 - STEEL

The number of beams used in a structure should be determined by taking into account the following:

- Traffic may need to be maintained over the structure during a future redecking. The number and spacing of the beams should allow for future staged construction of a new deck.
- No structure should have less than four beams.
- The maximum beam spacing is limited to 15 feet.
- A cost comparison should be done between the different numbers of beams under consideration, using the procedure discussed in Section 2.2.7 Cost Comparison for Number of Beams. Included in the cost analysis should be any increase

in grade required by greater beam depths. If there is no appreciable cost difference among the different numbers of beams being considered, the greater number should be used to provide increased redundancy in the structure.

7.4 Design Requirements

7.4.1 Welded Girders

To facilitate handling, the unsupported length of member/compression flange width (L/b) ratio preferably should not exceed 90. If using an L/b ratio of 90 results in an uneconomical design, a ratio of up to 100 may be used with permission from the Engineer of Design.

Flanges for welded beams should be proportioned to give a flange width/flange thickness (b/t) ratio between 12 and 20 with a preferred ratio of 16. The minimum flange width and thickness should be 12 inches and 3/4", respectively. These limits are set to avoid either a very thin wide flange that will distort when welded to the web, or a very thick narrow flange that would be uneconomical to purchase or laterally unstable.

The Structural Designer must verify that the design does not incorporate the need for transverse butt-welded joints in areas where such are not allowed. Restrictions on the location of these splices are given in Appendix D Standard Notes Structural Steel. Locations of transverse butt-welded splices for flanges and webs are typically not shown on the contract drawings, but are located by the fabricator. The Structural Designer must also verify that the stress range in areas where the fabricator is allowed to make transverse butt-welded splices meet AASHTO LRFD fatigue criteria.

7.4.2 Rolled Beams

The use of cover plates should be avoided on rolled beams. Increasing the size of the rolled beam itself is usually more cost effective, considering the expensive shop fit-up and reduced fatigue life of cover plate attachment.

Commentary: Most fabricators believe that when a cover plate is required, a welded plate girder will be more cost-effective. The camber can be cut into the web (eliminating heat-cambering) and the flanges can be sized exactly to meet design requirements.

If a cover plate is required, it should be designed to be a minimum of 1/2" thick and no thicker than the flange to which it is welded. In order to facilitate welding, cover plates should be smaller than the flange width by a minimum of twice the required weld size plus 1/8". For example, for a 12 inch wide flange

and cover plate requiring a 5/16" fillet weld, the maximum cover plate width would be 11-1/4". Refer to Example 7-1.

Example 7-1 Maximum Cover Plate Width

Given: Flange Width = 12 inch
Required Fillet Weld = 5/16"

$$12 \cdot in - \left(2 \times \frac{5}{16} \cdot in \right) - \frac{1}{8} \cdot in = 11 \frac{1}{4} \cdot in$$

7.4.3 Fabrication Considerations

The Designer should consult with the Bridge Quality Assurance Team for information pertaining to fabrication requirements. Some specific issues of interest may be weldment design, fabrication tolerances, shipping limitations and cost, fabricator limitations, appropriate material usage, and protective coatings.

Fracture critical members should be avoided, if possible. These members are non-redundant, and require an increase of 5% on the load side of the equation. They may need more frequent maintenance inspections. They may also be more expensive due to welding procedure requirements, base metal and weld testing, and construction inspection.

Weight considerations of the individual components should be carefully considered. By limiting maximum weight of fabricated continuous girder units, more steel fabrication shops can compete for the work. A brief review of the lifting capacity potential of fabrication shops may be prudent.

The Structural Designer should provide an optional field splice for simple span bridges exceeding 120 feet, or for even shorter spans where there is a challenging delivery route or constricted bridge site. A complete design of the optional splice should be shown on the plans.

On single span rolled beam structures with a camber of 3 inches or greater, fabricators should be given the option of fabricating welded plate girders in place of the rolled beams shown on the plans. The fabricator is responsible for determining the plate thicknesses based upon the depth and moment of inertia of the rolled section. This should be shown on the plans using the note given in Appendix D Standard Notes Structural Steel.

A beam stress diagram should be included on the plans of all continuous steel structures. It should indicate where the flanges are subject to tensile stress or stress reversal.

On rehabilitation projects that may require welding to existing steel, the Structural Designer must consider the weldability of the existing base metal. The Bridge Quality Assurance Team is an excellent resource for this information.

7.4.4 Diaphragms and Cross Frames

Diaphragms must be located within 5 feet of each point of dead load contraflexure on multiple span continuous structures with more than 250 yd³ of deck concrete. These diaphragms will allow for the construction of a slab joint. Diaphragms at these locations must be marked with an asterisk (*) and the appropriate note from Appendix D Standard Notes Structural Steel must be included on the plans. Cross frames and diaphragms should be specified on the plans in accordance with Standard Details 504 (15-22).

7.4.4.1 Instructions for Use of Standard Details

When selecting a diaphragm or cross frame, the span is defined as the distance between beams as measured along the diaphragm or cross frame. For beam depths and/or spans not covered in the following tables, the Structural Designer may make exceptions to the above limitations in special cases, providing that the design criteria are adequately satisfied. For cases where the design criteria are not adequately satisfied, the Structural Designer should design a special diaphragm or cross frame meeting the design requirements.

A. Slab Ends and Slab Joints

The diaphragms intended for use at slab ends and slab joints are listed in Table 7-2.

Table 7-2 End Diaphragms

	Beam Type	Beam Depth	Maximum Span (ft)
Type A1	Rolled	Any	12
Type A2	Rolled	Any	15
Type B	Welded	3'-0" to 3'-8"	20
Type C1	Welded	3'-8" to 3'-11"	15
Type C2	Welded	3'-11" to 4'-6"	20
Type D	Welded	4'-6" to 9'-0"	15

Commentary: The maximum span for all types except Type D is controlled by the moment strength of the member. The members were assumed to be simply supported beams loaded with two 20 k wheel loads plus impact 4 feet apart. The moment capacity of skewed diaphragms can be reassessed with the distance between wheels increased as a function of the skew angle, if warranted by special case. Two rows of bolts are considered necessary to carry moment induced by lateral loads and to ensure adequate stability during construction. The maximum span for Type D is controlled by the l/r ratio of the bottom lateral ($l/r \leq 140$). The beam depths for all types except Type D are sized to meet the depth requirements specified in AASHTO LRFD Section 6.7.4 - Diaphragms and Cross Frames.

B. Intermediate Locations

The cross frames intended for intermediate locations are listed in Table 7-3. To ensure that the intersection point of Types G or J does not act like a hinge during construction, Type H or K shall be used in the exterior bays.

Table 7-3 Intermediate Cross Frames

	Beam Type	Beam Depth	Maximum Span (ft)
Type E	Rolled	Any	10'-6"
Type F	Rolled	Any	12'
Type G	Welded	3'-0" to 3'-8"	13'-6"
Type H	Welded	3'-0" to 3'-8"	15'
Type J	Welded	3'-8" to 5'-0"	15'
Type K	Welded	3'-8" to 5'-0"	15'
Type L	Welded	5'-0" to 9'-0"	15'
Type M	Welded	5'-0" to 9'-0"	15'

Commentary: The maximum span for these types is controlled by the l/r ratio of the members ($l/r \leq 140$). The bolts in each connection are limited to the number considered to be adequate to transfer lateral loads to the slab and to distribute vertical loads during construction to insure stability.

7.4.5 Stiffeners and Diaphragm Connection Plates

7.4.5.1 General

For rolled beam designs, bearing stiffeners at end bearings and intermediate stiffeners should be used only when required by AASHTO

LRFD. At bearings other than end bearings, bearing stiffeners should always be used whether required by AASHTO LRFD or not.

For welded girder designs, bearing stiffeners should be used at all bearings. At bearings other than end bearings, a minimum of 2 intermediate stiffeners should be used at either side of the bearing stiffeners for a total of 4. Additional intermediate stiffeners should be used where required by AASHTO LRFD. On exterior beams, the intermediate stiffeners should be placed on the interior face of the web.

At bearings other than end bearings, on the fascia side of both rolled and welded exterior beams, either a single bearing stiffener placed at the centerline of bearing or two bearing stiffeners placed symmetrically on either side of the centerline of bearing should be used, at the option of the Structural Designer. If two bearing stiffeners are used, they must be a minimum of 8 inches apart to allow adequate access for welding. On the interior face of exterior beams, a stiffener layout as shown in Detail D or E in Figure 7-4 should be used as applicable.

7.4.5.2 Effect of Skew

Intermediate diaphragms and corresponding connection plates should be skewed on bridges with a 20° skew or less. On bridges with more than a 20° skew, the intermediate diaphragms and connection plates should be kept normal to the beams and arranged in a staggered pattern.

End bearing stiffeners should be skewed on bridges with a 30° skew or less, and used as diaphragm connection plates (refer to Detail A, Figure 7-3). On bridges with skews between 30° and 45°, the end bearing stiffeners should be kept normal to the beams with skewed connection plates for the end diaphragm connections (refer to Detail B, Figure 7-3). When the skew exceeds 45°, the end bearing stiffeners should be kept normal to the beams with bent connection plates for the end diaphragm connections (refer to Detail C, Figure 7-3).

When used as diaphragm connection plates, bearing stiffeners at all bearings other than end bearings should be skewed on bridges with a 45° skew or less (refer to Detail D, Figure 7-4). At all bearings other than end bearings on bridges with skews greater than 45°, two bearing stiffeners should be placed normal to the beams, and two bent stiffener plates used for the diaphragm connections (refer to Detail E, Figure 7-4).

Any bearing stiffener not used as a diaphragm connection plate should be kept normal to the beam centerline.

Any dimension shown as indeterminate on Details A through E (e.g. “6 inches minimum”, “as required”, etc.) must be determined by the Structural Designer and shown on the design drawings.

Note 7 in Standard Detail 504 (22) calls for the connection plates and stiffeners to be welded to the web with a fillet weld on both sides. If this weld arrangement is used, the dimensions shown as 6 inches minimum on Detail D and E in Figure 7-4 must be detailed on the design drawings as not less than 8 inches. If the dimension must be between 8 inches and 6 inches, the weld arrangement shown on Detail D must be detailed on the design drawings. If the skew is greater than 45°, the weld arrangements as shown on Detail C, Figure 7-3 and Detail E, Figure 7-4 must be used.

On beams with wide flanges and large skews, the Structural Designer should consider increasing the width of the bearing sole plate so that no part of a bearing stiffener group (e.g. Detail D) is more than the flange plate thickness outside the sole plate.

7.4.6 Handhold Bars

Handhold bars should be used on all girder structures with web depths of at least 6 feet that are not easily accessible for inspection by the under-bridge crane. The crane can reach a maximum of 25 feet under a bridge deck. The handhold bars should be placed on both sides of interior girders and on the inside only of exterior girders. Refer to Standard Details 504 (23-24) for more information. The connection at girder ends should be made 10 feet plus or minus from the centerline of bearings. This will allow the web to resist the tension force and discourage the public from climbing the girders.

Stiffeners should be a minimum of 1/2” thick where the handhold bar is terminated or spliced. Termination and splicing of handhold bars should occur only at stiffeners.

Commentary: The minimum thickness of the stiffener requirement is based on tests conducted at the University of Maine. Testing showed that thinner plates would be stressed beyond yield.

The angle clip detail is unable to resist or transmit the design forces. This may require adding stiffeners to both sides of the web, even if stiffeners are required on only one side.

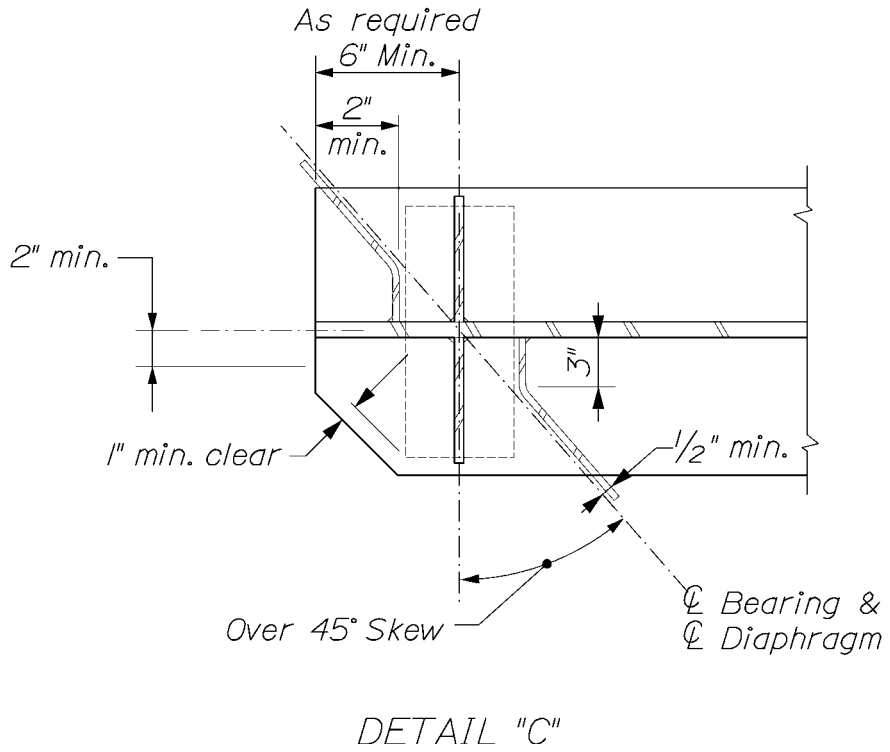
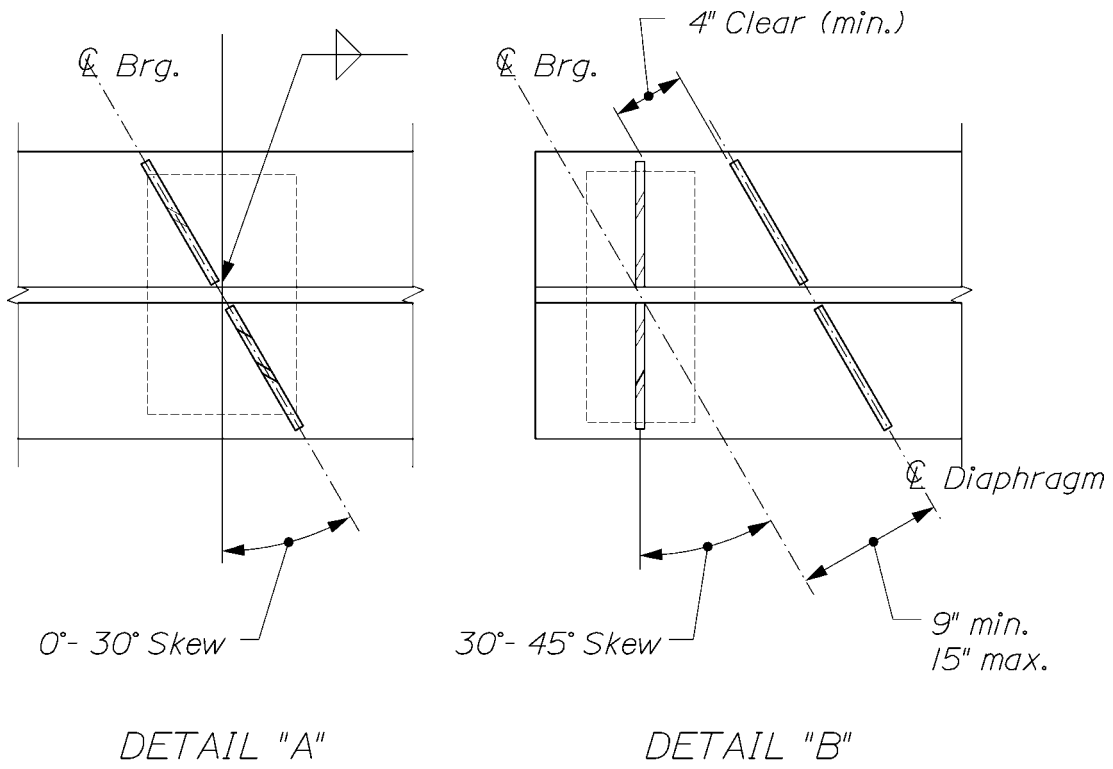


Figure 7-3 Stiffener/Connections Plate Details A Through C

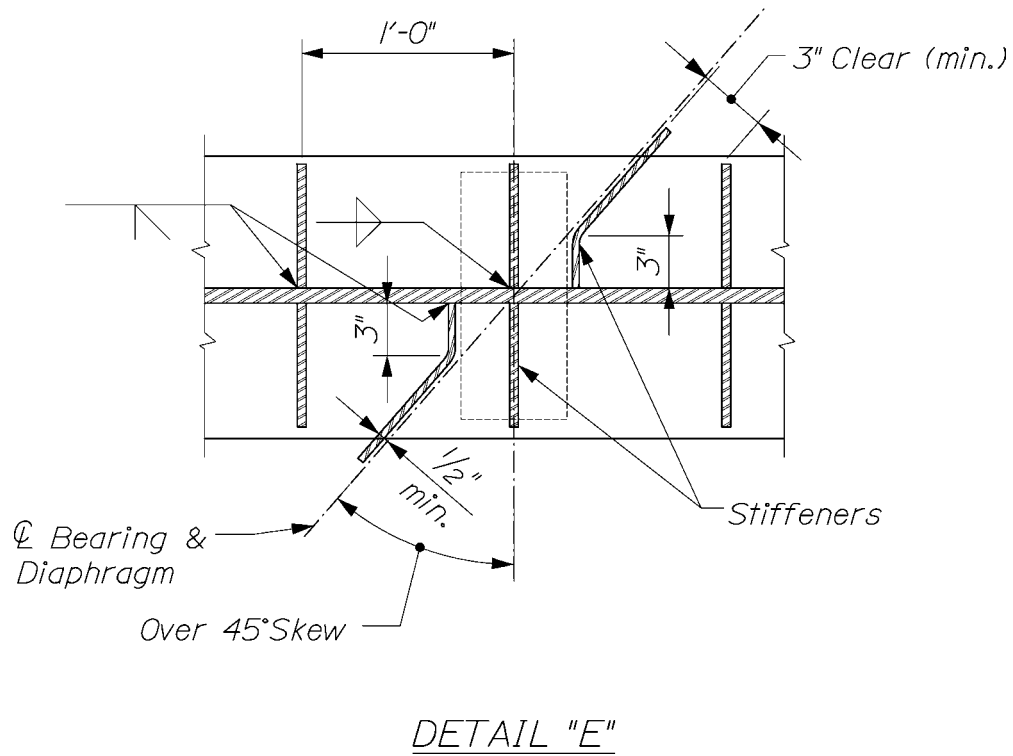
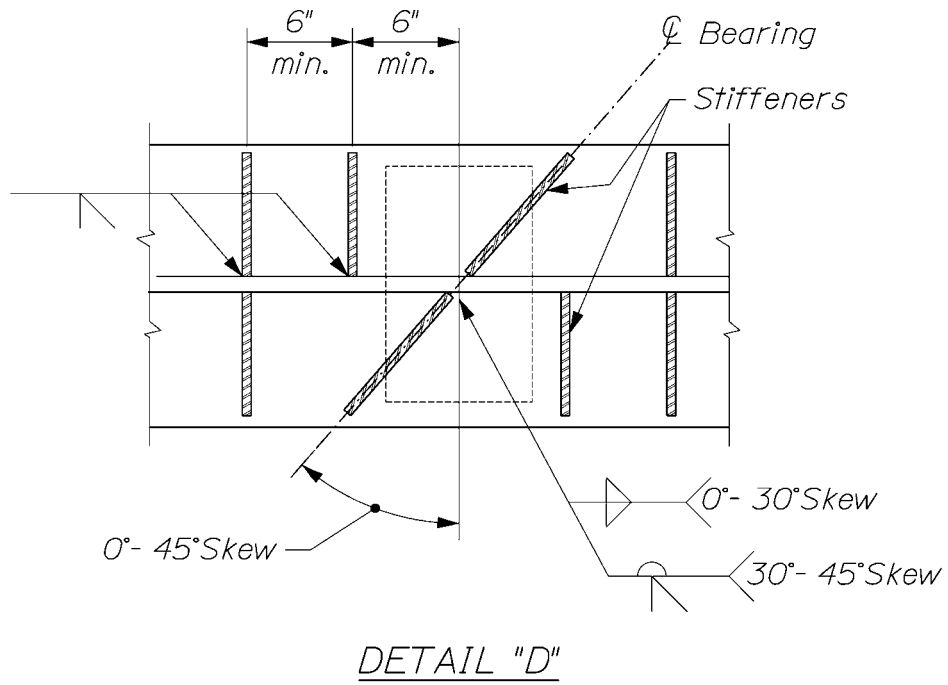


Figure 7-4 Stiffener/Connections Plate Details D and E

7.4.7 Slab Overhang Limits

In order to prevent excessive torsional deflections in beams during placement of the deck concrete, the slab overhang should not exceed the applicable value from Table 7-4. For overhangs exceeding the limits of Table 7-4, a torsional analysis of the exterior beam should be completed. Torsional analysis of the exterior beam should also be completed on all deck replacement and widening projects. As part of the shop drawing submittal, the Bridge Quality Assurance Team will complete a torsional analysis of the exterior beam for construction loading.

Table 7-4 Slab Overhang Limits

Beam Spacing	Maximum Overhang is the lesser of:
Less than 9'-0"	3'-0" or depth of beam
9'-0" to 10'-6"	1/3 of the beam spacing or depth of beam
Greater than 10'-6"	3'-6" or depth of beam

Note: Table 7-4 is for use on straight bridges. Maximum overhang for bridges with curved fascias is limited to 3'-6", or depth of beam plus 6", whichever is less.

7.4.8 Composite Design

All new steel girder bridges should be designed as composite structures.

References

AASHTO, 1978, and Interims through 1986, *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*

AASHTO, 1993, *Guide Specification for Horizontally Curved Highway Bridges*, Washington, DC

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Chapter 8

BURIED STRUCTURES



Curtis Bridge, Augusta



Twin # 2 Bridge, Farmington

8	BURIED STRUCTURES.....	8-1
8.1	General.....	8-1
8.1.1	Design	8-1
8.1.2	Construction Practices.....	8-1
8.2	Structural Plate Pipes and Pipe Arches	8-2
8.2.1	Design	8-2
8.2.2	Foundation Preparation	8-3
8.2.2.1	Bedrock.....	8-3
8.2.2.2	Soft Soils	8-4
8.2.3	Soil Envelope	8-4
8.2.4	Structural Plate Pipes and Pipe Arches Design Tables.....	8-7
8.2.5	End Treatment.....	8-14
8.2.5.1	Steel Structural Plate Structures	8-14
8.2.5.2	Aluminum Structural Plate Structures.....	8-15
8.3	Boxes.....	8-19
8.3.1	Design	8-19
8.3.2	Metal Structural Plate Box Culvert (Steel or Aluminum)	8-19
8.3.2.1	Foundation Preparation and Backfill Envelope.....	8-20
8.3.3	Concrete Box Culverts.....	8-21
8.4	Three-Sided Structures and Arches	8-23
8.4.1	Design	8-23
8.4.2	Metal Arches	8-23
8.4.2.1	Structural Plate Arch Tables.....	8-23
8.4.2.2	End Treatment.....	8-25
A.	Steel Structural Plate Arches	8-25
B.	Aluminum Structural Plate Arches	8-25
8.4.3	Concrete Rigid Frames and Arches.....	8-26
	References	8-27
	Table 8-1 Structural Steel Plate Pipes	8-8
	Table 8-2 Steel Structural Plate Pipe Arches.....	8-9
	Table 8-3 Steel Structural Plate Pipe Arches.....	8-10
	Table 8-4 Aluminum Structural Plate Pipes	8-11
	Table 8-5 Aluminum Structural Plate Pipe Arches	8-12
	Table 8-6 Precast Concrete Box Sizes	8-22
	Table 8-7 Steel Structural Plate Arches	8-24
	Table 8-8 Aluminum Structural Plate Arches	8-25
	Figure 8-1 Excavation and Backfill Limits for Pipe or Pipe Arch Structures	8-5
	Figure 8-2 Excavation and Backfill Limits for Pipe or Pipe Arch Structures - Soft Soils.....	8-6
	Figure 8-3 End Skews and Bevels for Pipe or Pipe Arch Structures.....	8-14
	Figure 8-4 End Reinforcement of Aluminum Pipe or Pipe Arch Structures	8-16
	Figure 8-5 Concrete Collar for Aluminum Pipe or Pipe Arch Structures - Plan and End Views.....	8-17

CHAPTER 8 – BURIED STRUCTURES

Figure 8-6 Concrete Collar for Aluminum Pipe or Pipe Arch Structures, Section
Views8-18

8 BURIED STRUCTURES

8.1 General

8.1.1 Design

A buried structure should be considered for any relatively short span crossing, if such a structure is hydraulically adequate for the site. These bridges or minor spans may be full culverts with a bottom or three sided structures founded on footings. The presence of bedrock, environmental impact concerns, or fish passage issues may preclude the use of a buried structure with a bottom.

All metal buried structures in tidal waters should be aluminum. In inland waters, steel is preferred due to lower initial cost, although aluminum should be used if the existing steel structure is being replaced after less than 50 years of service.

The invert for all culvert-type structures should normally be located at 12 inches below estimated normal streambed to accommodate fish passage. Reducing this depth to 6 inches below streambed may be considered with involvement from the Environmental Coordinator. Reducing the depth below streambed should only be considered when there are conflicts with utilities or bedrock. The material used inside the buried structure should be excavated native streambed with cost incidental to installation. The placement of the structure should follow the slope of the streambed when possible.

Backfill material should be Granular Borrow meeting the requirements of Subsection 703.19, Material for Underwater Backfill. Structural Excavation is considered incidental to the structural plate structure as stated in the Standard Specifications, while Granular Borrow is paid for separately.

Guardrail treatment for buried structures with shallow cover is shown in Standard Detail 606 (24).

The minimum cover should be checked at the face of guardrail from the top of the bituminous pavement. Refer to the design section for the structure type of interest for minimum cover requirements. On local roads, minimum cover requirements may be reduced to the amount specified by the manufacturer when warranted.

8.1.2 Construction Practices

Steel or aluminum structural plate structures should be constructed in the dry for the following reasons:

- o They are designed to interact with the surrounding soil, which offers structural support and load-carrying capacity. Therefore, it is important that the bedding material and the material within the backfill limits be sufficiently and uniformly compacted so that this soil-structure interaction will occur. This is especially true for pipe arches and metal boxes because it is important for the bedding material to accommodate the high corner pressures that develop in the structure.
- o Environmental concerns such as fish passage and erosion control require that the stream flow be maintained and that the stream be properly protected against siltation.
- o Saturated backfill could result in unbalanced buoyant forces.

To achieve dry construction conditions, a cofferdam should be used for the installation of all buried structures. For culvert-type structures, two cofferdam pay items should be added to the list of quantities, one for the upstream end and one for the downstream end. Three-sided structures will also need two cofferdams, one for each footing.

In some cases where 3 feet or more of ponded water exists, it may be acceptable to install structural plate pipes and pipe arches in the wet due to the environmental impacts of installing large cofferdams. However, these projects will be handled on a project-by-project basis, and will require close coordination with the regulating environmental agencies and the Construction Resident for approval of the installation procedures. Construction of metal box structures and pipe arches with 4 tsf corner pressure will always require a cofferdam.

Standard Specification Section 509 – Structural Plate Pipes, Pipe Arches, Arches, and Metals Box Culverts describes requirements for lift thickness and balanced lift placement. However, construction requirements controlling compaction of the soil envelope and bedding material are not currently in the Standard Specifications, so compaction requirements must be specified by a special provision or a note on the plans. See the Geotechnical Designer for the appropriate compaction specification.

8.2 Structural Plate Pipes and Pipe Arches

8.2.1 Design

Pipe and pipe arch sizes are determined by hydraulic analysis utilizing the invert at estimated normal streambed. Pipe arches are then increased to the next larger size when inverts are placed at 6 inches below streambed or two sizes larger when placed at 12 inches below streambed to offset the loss of

flow area. Round pipes are increased one size larger for both 6 inch and 12 inch placements of inverts below streambed.

Structural plate pipe arches should be selected for a maximum corner pressure of 2 tsf wherever possible. If a corner pressure of 4 tsf is necessary to select a pipe arch, refer to Section 8.2.3 Soil Envelope.

The lengths of pipes and pipe arches along bottom centerline of structure are specified in intervals of 2 feet. Span and rise dimensions are specified to the nearest 1-1/2”.

The minimum cover required is the span divided by 8 or 1'-6” minimum, whichever is greater.

8.2.2 Foundation Preparation

The bearing capacity of the foundation/backfill soils at the structure corners must be adequate to carry the design corner bearing pressure. Minimum excavation and underwater backfill bedding limits, regardless of the subgrade material, are shown in Figure 8-1.

In general, the foundation must provide uniform support for the pipe invert. Boulders, rocks, and/or soft spots must be excavated and voids backfilled with compacted underwater backfill material.

It is desirable for metal pipes and arches to bear on a relatively unyielding or fixed foundation as compared to the adjacent backfill. Otherwise, differential settlement between the soil envelope and the structure could create downdrag forces on the sidewalls of the pipe or arch. Uniform settlement along the length of the pipe is also desirable.

8.2.2.1 Bedrock

It is undesirable for metal pipes and pipe arches to bear directly on bedrock due to localized contact stresses. If bedrock is expected to be present at the bearing elevation, subgrade preparation may include:

- Removal of 1 foot minimum of bedrock and replacement with compacted underwater backfill material. The excavation should extend laterally 1 foot minimum beyond the proposed footprint of the pipe.
- Reshaping of the bedrock into a shallow v-shaped cradle in which to set the pipe.

- Placing the pipe on a smooth bedrock surface and placing flowable fill to provide even support in the haunch zone. This approach is the least desirable.

8.2.2.2 Soft Soils

Where poor or soft soils are encountered, consideration should be given to removing some or all of the poor material and replacing it with compacted underwater backfill material. A separation geotextile may be required to prevent mixing and migration of the underwater backfill material into the underlying poor soils.

For soils where a 1 foot thick bed will not support the design corner pressure, the bearing capacity of the foundation soil will need to be improved by removal of 2 feet of soft subgrade soil and replacement with compacted underwater backfill material and/or the use of a stabilization/reinforcement geotextile.

The structural backfill soil envelope should not settle more than the pipe, in order to avoid potential downdrag loads on the pipe. Therefore, any layer of stabilization/reinforcement geotextile should extend at least 1 foot beyond the pipe and into the soil backfill envelope.

8.2.3 Soil Envelope

The bearing material around the corners of the pipe and pipe arch must be capable of supporting the design corner pressure. The lateral limits of the pipe/pipe arch soil envelope are shown in Figure 8-1 and Figure 8-2. For structures with a corner pressure of 4 tsf, backfill limits should be increased to 1/2 span plus 6 feet each side.

For adequate compaction between pipes, the spacing of multiple pipes or pipe arches must be greater than 1/2 span or 3.0 feet, whichever is less. Structures backfilled with flowable fill will need to be restrained against flotation.

Structure backfill material and compaction requirements should be provided by the Geotechnical Designer for inclusion in the contract documents.

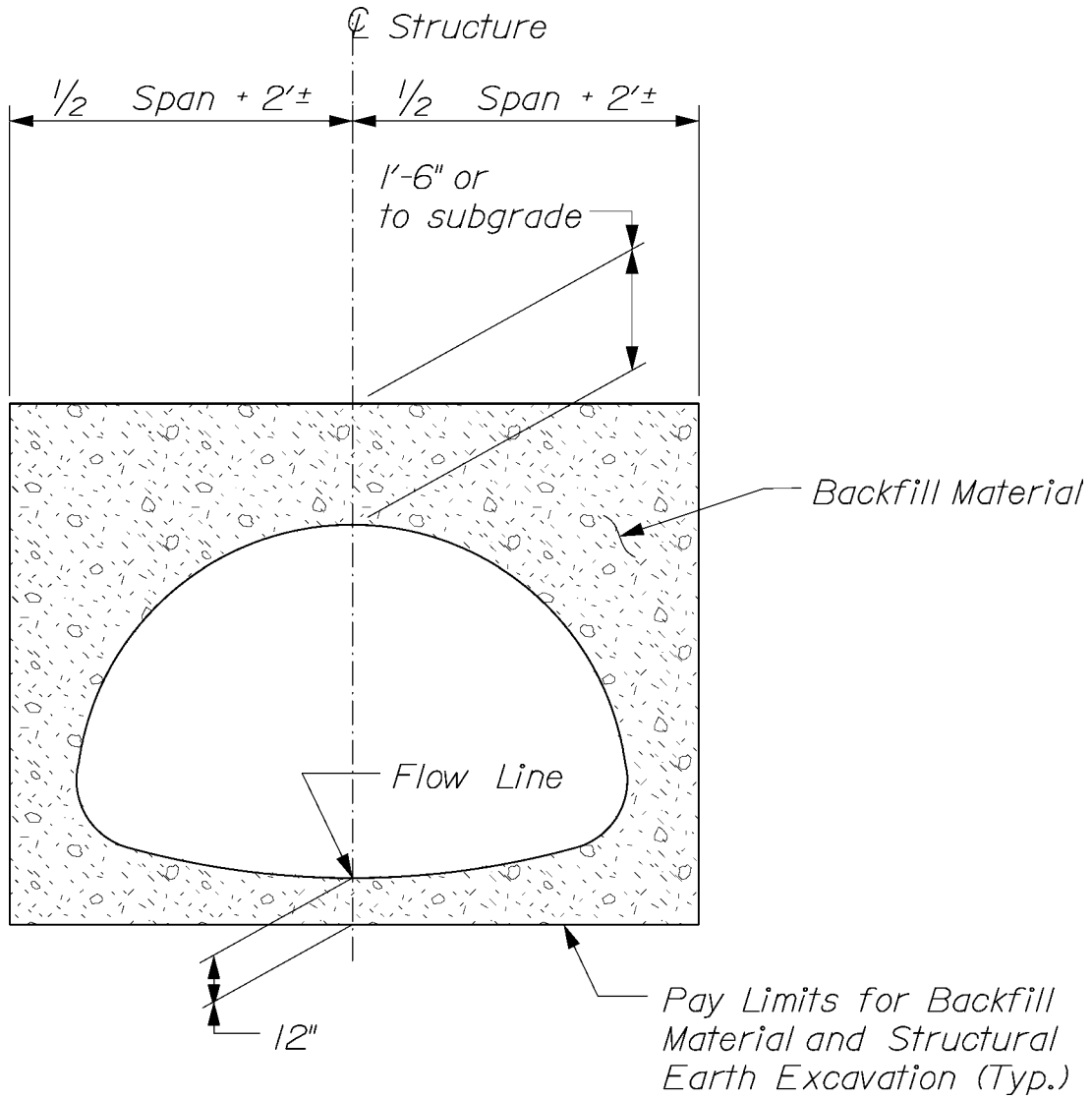


Figure 8-1 Excavation and Backfill Limits for Pipe or Pipe Arch Structures

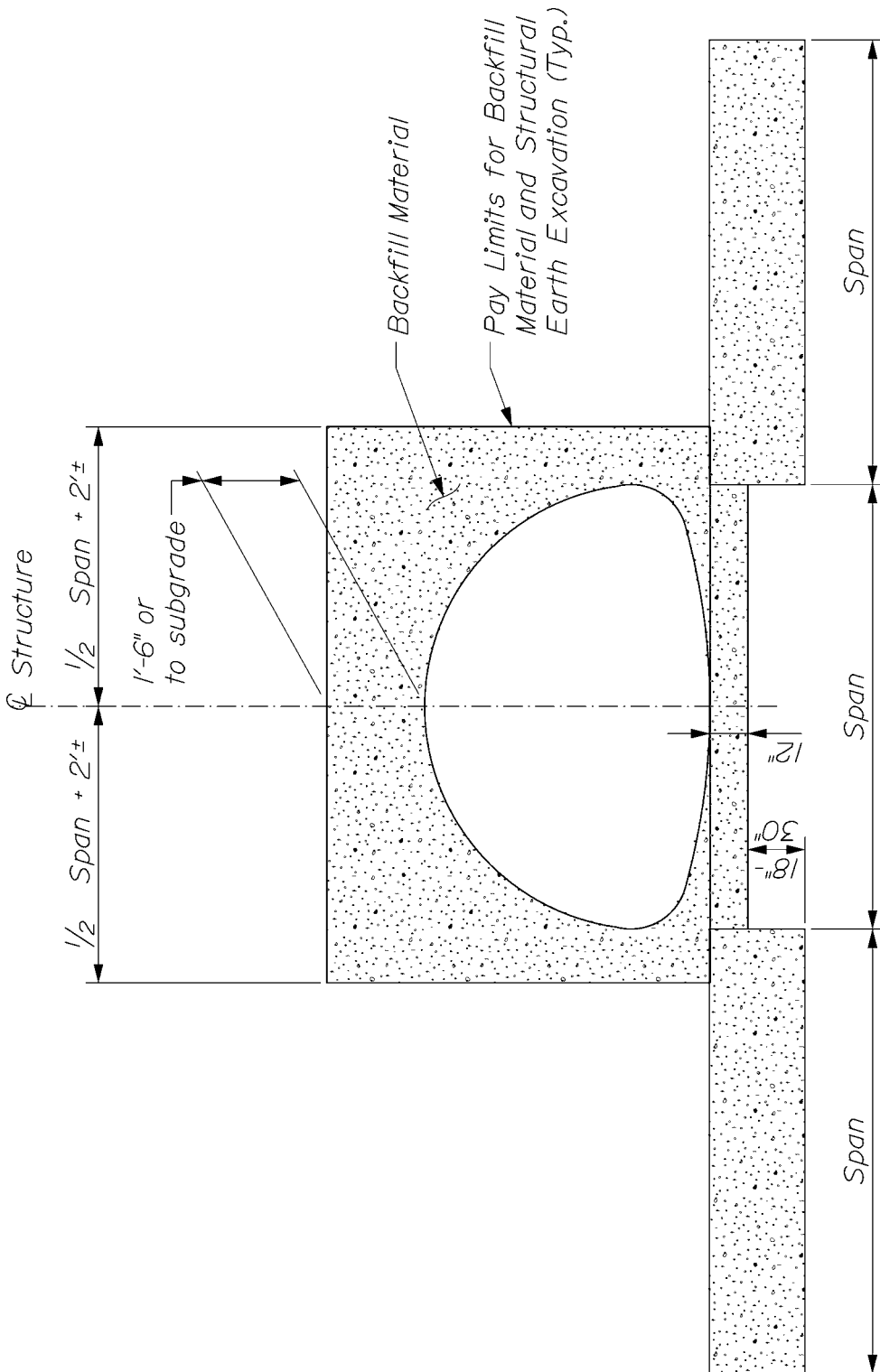


Figure 8-2 Excavation and Backfill Limits for Pipe or Pipe Arch Structures - Soft Soils

8.2.4 Structural Plate Pipes and Pipe Arches Design Tables

These tables specify structural plate thickness requirements for a given structure size, corner radius, and corner radius pressure up to a maximum fill height of 30 feet. Additional metal thickness to resist abrasion and corrosion has been included in these tables.

All steel plates below ordinary high water should be specified two available plate thicknesses heavier than those shown in the tables. In stream crossings where corrosion or abrasion is known to be severe on metal pipes, consideration should be given to providing further increases in thickness over that indicated in the tables.

The plate thickness for aluminum structural plate structures should be specified on the contract documents as shown in the tables. If reinforcing ribs are required for the structure, they should be designed by the manufacturer. The thickness of the plates for design should be the thickness stated on the plans minus 0.055 inches. Refer to Appendix D Standard Notes.

Commentary: Some readily available structural plate pipe and pipe arch sizes have changed from those listed in the 1996 Bridge Design Manual, and are reflected in the tables in this Guide. It should be noted however, that all previously available sizes could be obtained if needed for an extension of an existing structure.

A computer program was developed to design all available structural plate pipes based on AASHTO Design Criteria according to Section 9 of AASHTO 1977 Specifications through interims 1981. Pipe tables were developed along with the computer program to provide a documented design and detail guide for structural plate pipes, pipe arches, and plate arches. The design criteria for this analysis are as follows:

Live Load = HS25

Weight of Earth = 125 pcf

Soil Modulus (E') = 1050 psi

Compaction = 85% Standard Density (AASHTO T99)

Safety Factor for Wall Buckling = 2 for pipes and pipe arches, 4 for plate arches

Bolts (aluminum and steel pipes) = Galvanized Steel

The metal thickness shown in the steel pipe tables was derived by providing an additional 0.060" to the minimum design requirements, from the computer input, and rounding up to the nearest available plate thickness. This provides a reserve thickness for abrasion and corrosion losses in addition to the added thickness for plates below ordinary high water.

The metal thickness shown in the aluminum pipe tables was derived by providing an additional 0.055" to the minimum design requirements, from the computer output, and rounding up to the nearest available plate thickness. This provides a reserve thickness for abrasion and corrosion losses and provides additional stiffness for handling.

Documentation of all design data is located in the Bridge Program Library Research Files. These pipe tables apply to highway fills above the top of pipes up to 30 feet in depth, though computer output data is available for higher fill depths. Plate thickness for fill heights greater than 30 feet must be approved by the Engineer of Design.

Table 8-1 Structural Steel Plate Pipes

Pipe Diameter (in)	Area (ft ²)	Min. Cover (ft)	Maximum Fill Height Above Top of Pipe (feet) Metal Thickness in inches					
			0.138 in	0.168 in	0.188 in	0.218 in	0.249 in	0.280 in
60	20	1.5	20	30	30	30	30	30
66	24	1.5	20	30	30	30	30	30
72	28	1.5	18	30	30	30	30	30
78	33	1.5	16	30	30	30	30	30
84	38	1.5	16	30	30	30	30	30
90	44	1.5	14	25	30	30	30	30
96	50	1.5	14	25	30	30	30	30
102	57	1.5	12	25	30	30	30	30
108	64	1.5	12	20	30	30	30	30
114	71	1.5	12	20	30	30	30	30
120	78	1.5	11	20	25	30	30	30
126	87	1.5	10	20	25	30	30	30
132	95	1.5	9	20	25	30	30	30
138	104	1.5	9	18	25	30	30	30
144	113	1.5	8	18	20	30	30	30
150	123	1.6	8	16	20	30	30	30
156	133	1.6	7	16	20	30	30	30
162	143	1.7		16	20	25	30	30
168	154	1.8		14	20	25	30	30
174	165	1.8		14	20	25	30	30
180	177	1.9		14	18	25	30	30
186	189	1.9		12	18	25	30	30
192	201	2.0			18	20	30	30
198	214	2.1			16	20	30	30
204	227	2.1			16	20	30	30
210	241	2.2				20	25	30
216	254	2.3				20	25	30
222	269	2.3				20	25	30
228	284	2.4				20	25	30
234*	299	2.4					25	30
240*	314	2.5					25	30
246*	330	2.6					20	25
252*	346	2.6					20	25

* These pipes are not fabricated with available thicknesses to allow increasing bottom plates by 2 available sizes. Their use shall be only as approved by the Engineer of Design.

Table 8-2 Steel Structural Plate Pipe Arches
18 inch Corner Radius

Size Span x Rise	Area (ft ²)	Min. Cover (ft)	Max. Fill Height Above Top of Pipe Arch (feet) For Indicated Design Corner Pressure in tons/ft ² Metal Thickness in Inches			
			0.138 in		0.168 in	
			2 tsf	4 tsf	2 tsf	4 tsf
6'-1" x 4'-7"	22	1.5	14	18	14	30
6'-4" x 4'-9"	24	1.5	14	18	14	30
6'-9" x 4'-11"	26	1.5	14	16	14	25
7'-0" x 5'-1"	28	1.5	12	16	12	25
7'-3" x 5'-3"	31	1.5	12	14	12	25
7'-8" x 5'-5"	33	1.5	12	14	12	20
7'-11" x 5'-7"	35	1.5	12	14	12	20
8'-2" x 5'-9"	38	1.5	11	14	11	20
8'-7" x 5'-11"	40	1.5	11	12	11	20
8'-10" x 6'-1"	43	1.5	10	12	10	20
9'-4" x 6'-3"	46	1.5	10	12	10	20
9'-6" x 6'-5"	49	1.5	10	12	10	20
9'-9" x 6'-7"	52	1.5	9	11	9	18
10'-3" x 6'-9"	55	1.5	8	11	8	18
10'-8" x 6'-11"	58	1.5	7	10	7	16
10'-11" x 7'-1"	61	1.5	7	9	7	16
11'-5" x 7'-3"	64	1.5	6	9	6	16
11'-7" x 7'-5"	67	1.5	6	9	6	16
11'-10" x 7'-7"	71	1.5	6	8	6	16
12'-4" x 7'-9"	74	1.5	6	8	6	14
12'-6" x 7'-11"	78	1.6	6	8	6	14
12'-8" x 8'-1"	81	1.6	6	8	6	14
12'-10" x 8'-4"	85	1.6	6	7	6	14

In general, use 2 tsf design corner pressure. For 4 tsf design corner pressure, refer to Section 8.1.1 Design.

Table 8-3 Steel Structural Plate Pipe Arches
31 inch Corner Radius

Size Span x Rise	Area (ft ²)	Min. Cover (ft)	Max. Fill Height Above Top of Pipe Arch (feet) For Indicated Design Corner Pressure in tons/ft ² Metal Thickness in inches							
			0.138 in		0.168 in		0.188 in		0.218 in	
			2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf
13'-3"x 9'-4"	98	1.7	7	7	11	16	11	20	11	20
13'-6"x 9'-6"	102	1.7	7	7	11	16	11	20	11	20
14'-0"x 9'-8"	106	1.8	6	6	11	14	11	20	11	20
14'-2"x 9'-10"	110	1.8	6	6	11	14	11	20	11	20
14'-5"x 10'-0"	115	1.8	6	6	11	14	11	20	11	20
14'-11"x 10'-2"	119	1.9	6	6	10	14	10	18	10	20
15'-4"x 10'-4"	124	1.9	6	6	10	14	10	18	10	20
15'-7"x 10'-6"	129	1.9	6	6	9	12	9	18	9	20
15'-10"x 10'-8"	133	2.0	6	6	9	12	9	18	9	20
16'-3"x 10'-10"	138	2.0	5	5	9	12	9	16	9	20
16'-6"x 11'-0"	143	2.1			9	12	9	16	9	20
17'-0"x 11'-2"	148	2.1			8	12	8	16	8	18
17'-2"x 11'-4"	153	2.1			8	12	8	16	8	18
17'-5"x 11'-6"	158	2.2			8	12	8	16	8	18
17'-11"x 11'-8"	163	2.2			8	11	8	14	8	18
18'-1"x 11'-10"	168	2.3			8	11	8	14	8	18
18'-7"x 12'-0"	174	2.3			8	11	8	14	8	16
18'-9"x 12'-2"	179	2.3			7	11	7	14	7	16
19'-3"x 12'-4"	185	2.4			7	10	7	14	7	16
19'-6"x 12'-6"	191	2.4					7	14	7	16
19'-8"x 12'-8"	196	2.5					7	14	7	16
19'-11"x 12'-10"	202	2.5					7	14	7	16
20'-5"x 13'-0"	206	2.6					7	12	7	14
20'-7"x 13'-2"	214	2.6					7	12	7	14

In general, use 2 tsf design corner pressure. For 4 tsf design corner pressure, refer to Section 8.1.1 Design.

Table 8-4 Aluminum Structural Plate Pipes

Pipe Diameter (in)	Area (ft ²)	Min. Cover (ft)	Maximum Fill Height Above Top of Pipe (feet) Metal Thickness in inches				
			0.150 in	0.175 in	0.200 in	0.225 in	0.250 in
60	20	1.5	25	30	30	30	30
66	24	1.5	20	30	30	30	30
72	28	1.5	20	30	30	30	30
78	33	1.5	20	30	30	30	30
84	38	1.5	18	25	30	30	30
90	44	1.5	18	25	30	30	30
96	50	1.5	16	25	30	30	30
102	57	1.5	14	20	30	30	30
108	64	1.5	14	20	30	30	30
114	71	1.5	14	20	25	30	30
120	78	1.5	12	20	25	30	30
126	87	1.5	12	18	25	30	30
132	95	1.5	12	18	20	30	30
138	104	1.5	11	16	20	25	30
144	113	1.5	10	16	20	25	30
150	123	1.6		16	20	25	30
156	133	1.6		14	20	25	25
162	143	1.7		14	20	20	25
168	154	1.8			18	20	25
174	165	1.8			18	20	25
180	177	1.9			18	20	25
186	189	1.9				20	20
192	201	2.0				20	20
198	214	2.1				20	20
204	227	2.1					20
210	241	2.2					20
216*	254	2.3					20
222*	269	2.3					20
228*	284	2.4					20
234*	299	2.4					20
240*	314	2.5					20

* These pipes do not have the reserve thickness provisions as the other pipes. Their use will be only as approved by the Engineer of Design.

Table 8-5 Aluminum Structural Plate Pipe Arches

Size Span x Rise	Area (ft ²)	Min. Cover (ft)	Maximum Fill Height Above Top of Pipe Arch (feet) For Indicated Corner Pressure in tons/ft ² Metal Thickness in inches									
			0.150 in		0.175 in		0.200 in		0.225 in		0.250 in	
			2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf
6'-7" x 5'-8"	30	1.5	20	20	20	30	20	30	20	30	20	30
6'-11" x 5'-9"	32	1.5	18	18	20	25	20	30	20	30	20	30
7'-3" x 5'-11"	34	1.5	18	18	20	25	20	30	20	30	20	30
7'-9" x 6'-0"	37	1.5	16	16	18	25	18	30	18	30	18	20
8'-1" x 6'-1"	39	1.5	16	16	18	25	18	30	18	30	18	30
8'-5" x 6'-3"	42	1.5	16	16	18	20	18	30	18	30	18	30
8'-10" x 6'-4"	44	1.5	14	14	16	20	16	30	16	30	16	30
9'-3" x 6'-5"	47	1.5	14	14	14	20	14	25	14	30	14	30
9'-7" x 6'-6"	50	1.5	14	14	14	20	14	25	14	30	14	30
9'-11" x 6'-8"	53	1.5	12	12	14	20	14	25	14	30	14	30
10'-3" x 6'-9"	55	1.5	12	12	14	20	14	25	14	25	14	30
10'-9" x 6'-10"	58	1.5	12	12	12	18	12	25	12	25	12	25
11'-1" x 7'-0"	61	1.5	12	12	12	18	12	20	12	25	12	25
11'-5" x 7'-1"	64	1.5	11	11	12	16	12	20	12	25	12	25
11'-9" x 7'-2"	67	1.5	10	10	12	16	12	20	12	25	12	25
12'-3" x 7'-3"	70	1.5	10	10	10	16	10	20	10	20	10	20
12'-7" x 7'-5"	74	1.6	9	9	10	16	10	20	10	20	10	20
12'-11" x 7'-6"	77	1.6	9	9	10	14	10	20	10	20	10	20
13'-1" x 8'-2"	83	1.6	9	9	10	14	10	20	10	20	10	20
13'-1" x 8'-4"	87	1.6	9	9	10	14	10	20	10	20	10	20
13'-11" x 8'-5"	90	1.7	8	8	9	14	9	20	9	20	9	20
14'-0" x 8'-7"	94	1.8	8	8	10	14	10	18	10	20	10	20
13'-11" x 9'-5"	101	1.7	8	8	11	14	11	18	11	20	11	20

(This table continues on the next page.)

Size Span x Rise	Area (ft ²)	Min. Cover (ft)	Maximum Fill Height Above Top of Pipe Arch (feet) For Indicated Corner Pressure in tons/ft ² Metal Thickness in inches									
			0.150 in		0.175 in		0.200 in		0.225 in		0.250 in	
			2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf	2 tsf	4 tsf
14'-3" x 9'-7"	106	1.8	8	8	11	11	11	14	11	20	11	20
14'-8" x 9'-8"	110	1.8	8	8	10	11	10	14	10	20	10	20
14'-11" x 9'-10"	114	1.9			10	11	10	14	10	20	10	20
15'-4" x 10'-0"	119	1.9			10	11	10	14	10	20	10	20
15'-7" x 10'-2"	123	1.9			9	11	9	14	9	20	9	20
16'-1" x 10'-4"	128	2.0			9	11	9	14	9	20	9	20
16'-4" x 10'-6"	132	2.0			9	11	9	14	9	20	9	20
16'-9" x 10'-8"	137	2.1					8	14	8	18	8	18
17'-0" x 10'-10"	142	2.1					8	14	8	18	8	18
17'-3" x 11'-0"	147	2.2					8	14	8	18	8	18
17'-9" x 11'-2"	152	2.2					8	14	8	18	8	18
18'-0" x 11'-4"	157	2.3					8	14	8	18	8	18
18'-5" x 11'-6"	162	2.3							8	16	8	16
18'-8" x 11'-8"	167	2.3							8	16	8	16
19'-2" x 11'-9"	172	2.4							7	14	7	16
19'-5" x 11'-11"	178	2.4							7	14	7	16
19'-10" x 12'-1"	183	2.5									7	16
20'-1" x 12'-3"	188	2.5									7	14
20'-1" x 12'-6"	194	2.5									7	14
20'-10" x 12'-7"	200	2.6									7	14
21'-6" x 12'-11"*	211	2.7									6	12

(This table is continued from previous page.)

1. In general, use 2 tsf design corner pressure. For 4 tsf design corner pressure, refer to 8.1.1 Design.
2. The pipe arch indicated with an (*) does not have the reserve thickness provision as the other pipe arches. Its use will be only as approved by the Engineer of Design.

8.2.5 End Treatment

8.2.5.1 Steel Structural Plate Structures

All pipes and pipe arches should have end bevels cut on a 1.75:1 slope normal to the end skew as shown in Figure 8-3. A bottom step cut should be provided as recommended by the manufacturer. The embankment slopes above the pipe should be 2:1, but may be steepened to 1.75:1 when warranted. Embankment slopes steeper than 2:1 must be stabilized by a layer of riprap with a minimum thickness of 1 foot.

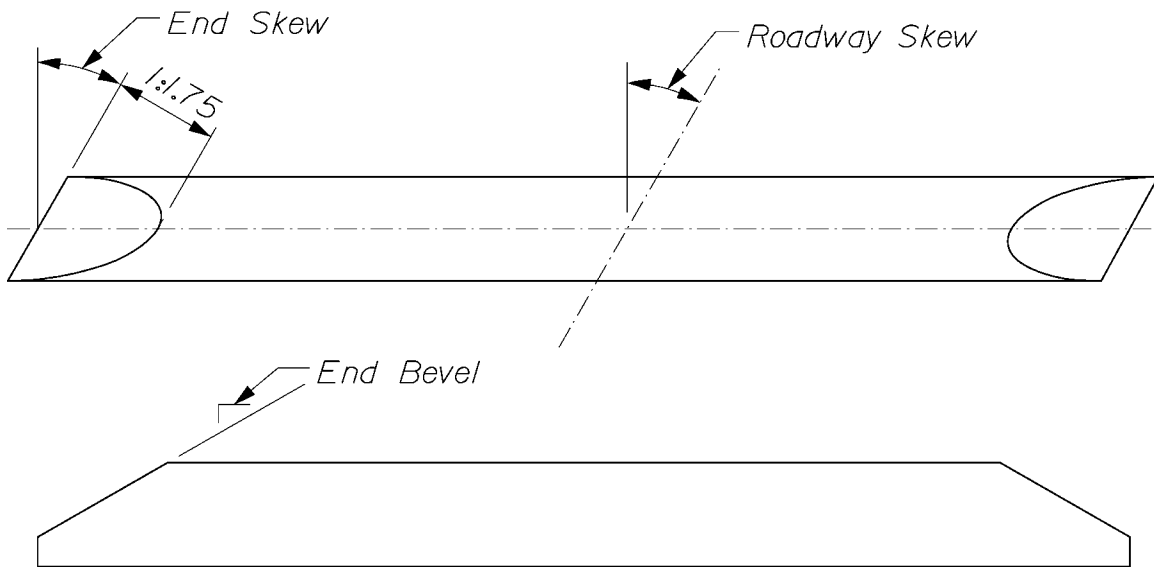


Figure 8-3 End Skews and Bevels for Pipe or Pipe Arch Structures

The treatment of the end skew will vary depending upon the roadway skew and the size of the structure. A pipe arch is considered “large” when the span is approximately 15 feet, and the rise is approximately 12 feet.

- o Roadway skew up to 15°:

Small pipe, and/or deep cover over pipe - Ends should be cut square and the embankment slopes warped to fit the pipe.

Large pipe with shallow cover over pipe - End skews should be provided to prevent warped slopes from being steeper than 1.5:1.

- o Roadway skew between 15° and 20° inclusive:

Small pipe - End skews should be provided to prevent warped slopes from being steeper than 1.5:1.

Large pipe - Same as for small pipe, but the end skew is limited to a maximum of 15°.

- o Roadway skew over 20°:

End skews should be provided giving consideration to the size of the structure and the amount of cover and to prevent warped slopes from being steeper than 1.5:1. End skew is limited to a maximum of 20°. In some cases, slight lengthening of the structure may be necessary to meet the above requirements.

8.2.5.2 Aluminum Structural Plate Structures

The end treatments described for steel structural plate structures also apply to aluminum pipes and pipe arches, except that the end skew should not exceed 15°. When used, the height of the bottom step cut should be approximately equal to 1/3 times the rise.

In addition, end reinforcement should be provided as shown in Figure 8-4 for all aluminum plate pipes and pipe arches with spans greater than 10 feet. Where end reinforcement is not required, a top step cut should be provided with a height equal to about half the bottom step for pipe arches.

End reinforcement devices must be composed of aluminum with sufficient strength to provide a minimum section modulus about an axis perpendicular to the center of the pipe of 1.10 in³/ft of pipe circumference. Maximum spacing of the devices is 5'-5", with attachments using 3/4" diameter aluminum bolts. Section properties, details of the device, and the method of attachment must be submitted to the Resident for approval. Refer to Figure 8-4 and Appendix D Standard Notes Structural Plate Structures.

In areas where ice is common, particularly on coastal streams, an alternate end reinforcement design utilizing a concrete collar should be considered. Refer to Figure 8-5 and Figure 8-6 for further guidance. In these cases, a barrier must be placed between the aluminum and the concrete to prevent interaction.

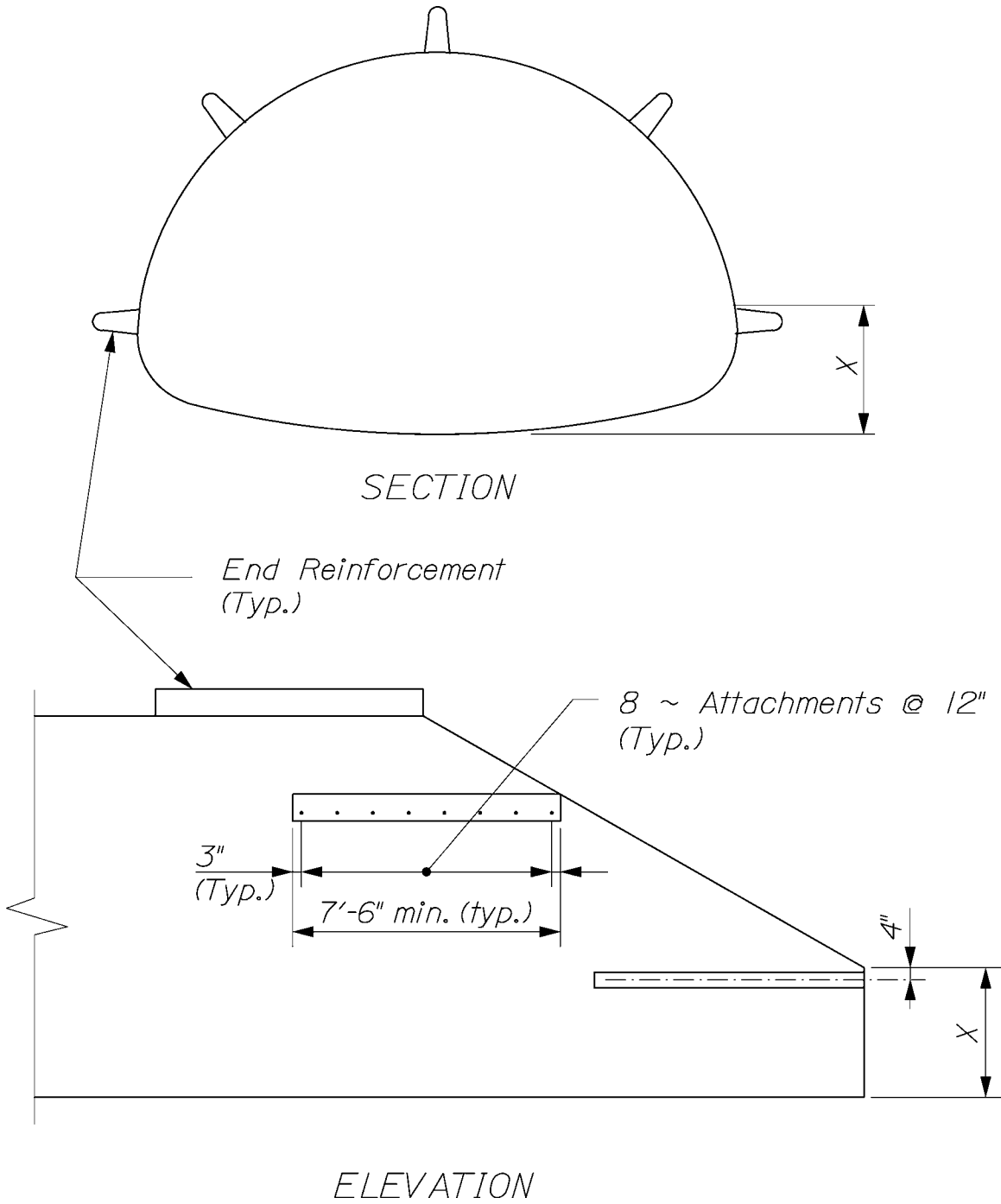


Figure 8-4 End Reinforcement of Aluminum Pipe or Pipe Arch Structures

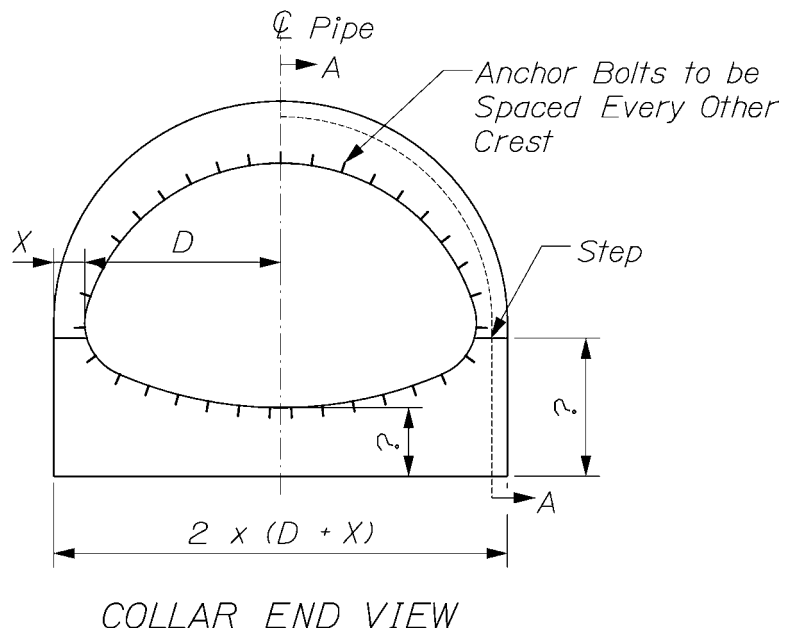
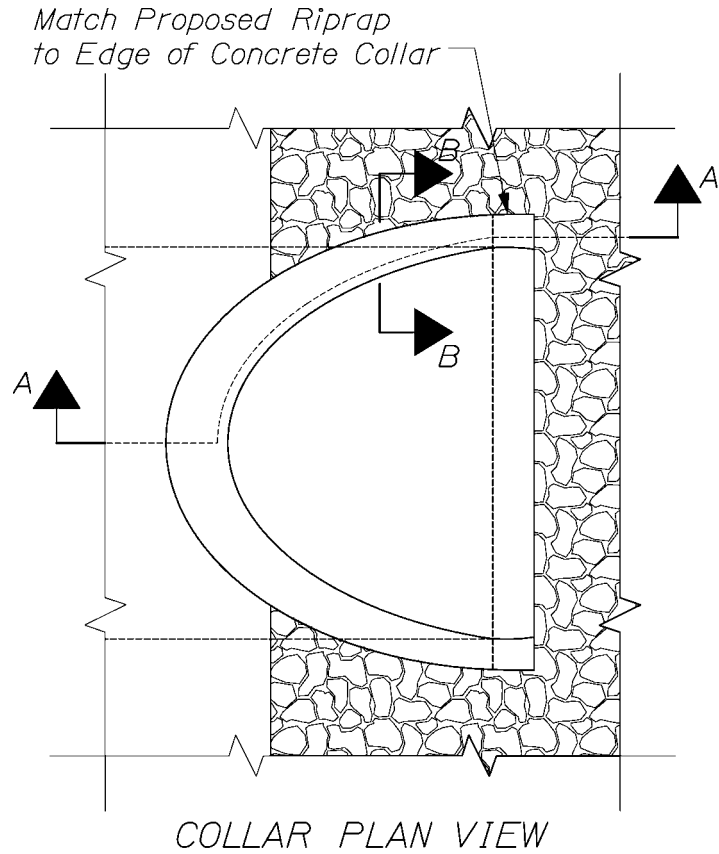


Figure 8-5 Concrete Collar for Aluminum Pipe or Pipe Arch Structures - Plan and End Views

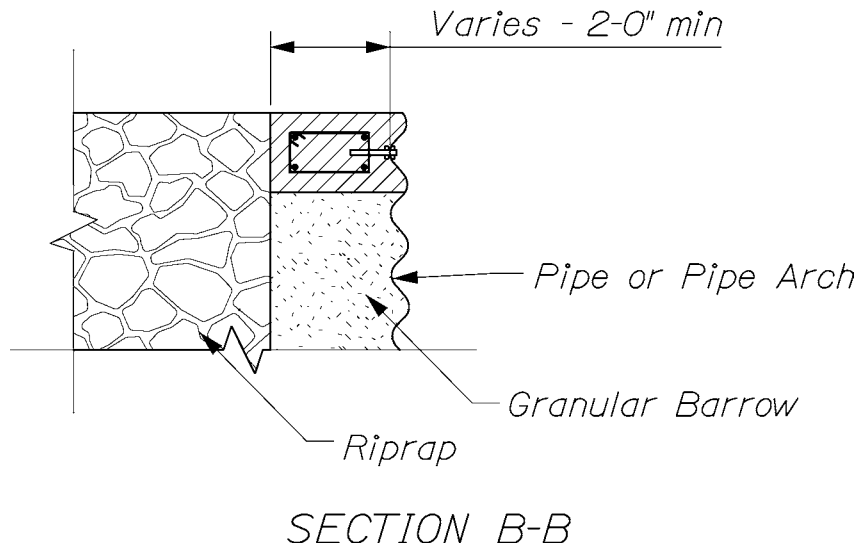
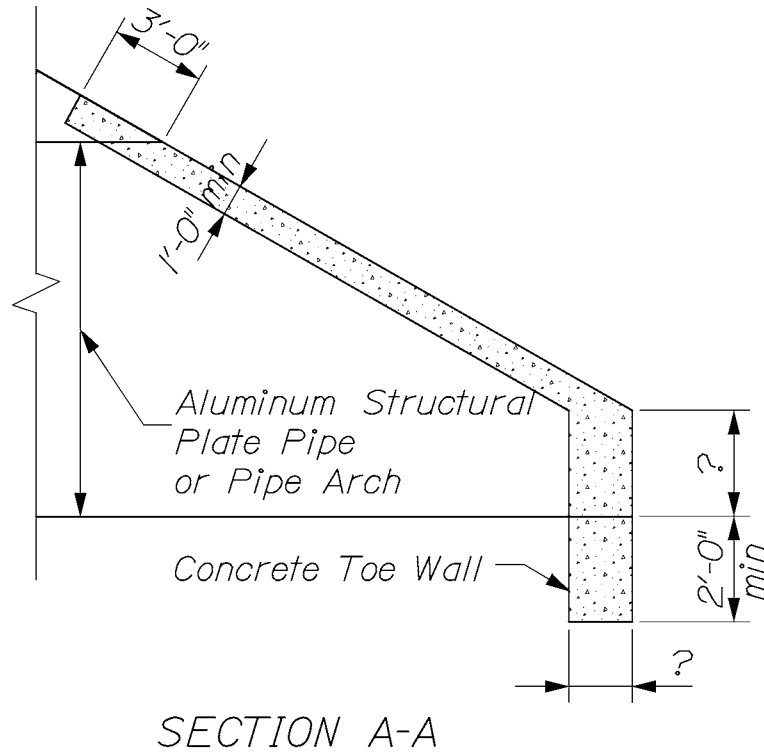


Figure 8-6 Concrete Collar for Aluminum Pipe or Pipe Arch Structures, Section Views

8.3 Boxes

8.3.1 Design

Box structures discussed here will have a bottom plate or slab. For those without a bottom, refer to Section 8.4 Three-Sided Structures and Arches. Material may be precast concrete, cast-in-place concrete, or metal structural plate. All box structures should include toe walls at both ends to prevent undermining.

The Designer should specify HS25 loading and allow the manufacturer to design the box structure. Only the basic layout and required hydraulic opening is detailed, so the Contractor can choose among available proprietary products.

If possible, the structure length should be designed long enough to preclude the need for wingwalls, using beveled ends similar to those used for pipe projects. Also, the ends of the box should not be skewed; the length of the structure should be increased slightly and the sideslopes warped to accommodate the skew of the box to the roadway. If wingwalls are required, refer to Section 5.6.5 Prefabricated Proprietary Walls.

8.3.2 Metal Structural Plate Box Culvert (Steel or Aluminum)

Generally, an aluminum structural plate box culvert is preferred over steel due to the uncertainty of the long term durability of the steel frame, and the potential for catastrophic failure when deterioration occurs. Toe walls can be metal or concrete, and should have a minimum height of 2 feet.

The plate thickness of the headwalls, wingwalls, and invert plates, should be as recommended by the manufacturer. The shell plate thickness should equal the plate thickness recommended by the manufacturer plus 0.055 inches or 0.060 inches for aluminum or steel structures, respectively. If reinforcing ribs are required for the structure, they should be designed by the manufacturer. The thickness of the plates for design should be the thickness stated on the plans minus 0.055 inches or 0.060 inches for aluminum or steel structures, respectively. Refer to Appendix D Standard Notes Structural Plate Structures.

Follow the manufacturer's recommendations in specifying length, which is usually in multiples of 4.5 feet to minimize structure costs. Use of headwalls will require further length restrictions, as discussed below.

When headwalls are used, the total structure length should be a multiple of 9 inches, and the headwall should be at least 3 feet away from the face of the

guardrail. This clearance will reduce the risk of headwall damage if the guardrail is hit. For shallow cover situations, the length of the shell may need to be increased so that the anchor rod between the shell and the headwall avoids the concrete around the guardrail posts. The anchor rods supporting the headwalls cannot be embedded in concrete.

It is recommended that roads be paved over metal boxes whenever possible. The minimum cover described by the manufacturer should be increased by 6 inches for a gravel road.

8.3.2.1 Foundation Preparation and Backfill Envelope

Structural plate box culverts must be founded on a 12 inch minimum leveling course consisting of compacted structural backfill, conforming to granular borrow for underwater backfill. In general, the foundation must provide adequate and uniform support for the box bottom and bedding material. Boulders or rocks within the limits of required bedding will need to be excavated and voids backfilled with bedding material. The subgrade foundation soils must be composed of stiff to hard in-situ soil, stabilized soil, or compacted fill.

A. Soft Soils

Where poor or soft soils are encountered, consideration should be given to removing some or all of the poor material and replacing it with compacted underwater backfill material. A separation geotextile may be required to prevent migration and mixing of the compacted underwater backfill with the underlying poor soils.

The bearing capacity of soft foundation soils may be improved with the use of a reinforcement/separation geotextile. Furthermore, the structural backfill soil envelope should not be allowed to settle more than the box, in order to avoid potential downdrag loads on the sidewalls. Therefore, the layer of reinforcement geotextile should extend at least 1 foot beyond the footprint of the box. Uniform settlement along the length of the box is desirable.

B. Soil Envelope

The lateral limit of the soil envelope is a minimum of 3 feet wide at the footing and should extend upward to the road surface elevation. Backfill requirements should be supplied by the Geotechnical Designer for inclusion in the contract documents.

Structures backfilled with flowable fill must be guarded against flotation.

8.3.3 Concrete Box Culverts

Concrete boxes can be precast or cast-in-place. If precast is chosen, standard sizes should be used whenever possible to save cost. However, non-standard sizes can be fabricated when needed. When using a non-standard size, the manufacturer should be contacted to be sure the sections are not too big or heavy to be transported to the site and erected.

Precast concrete boxes are detailed on contract plans with only the basic layout and required hydraulic opening, so that the Contractor can choose among available proprietary products. The manufacturer is responsible for the design of the structure, which includes determination of wall thickness, haunch thickness, and reinforcement. The loading specified for the structure should be HS25 and Special Provision 534 must be included in the PS&E package. Soil type 4 should be used in the design of earth loads from the soil envelope (refer to Section 3.6 Earth Loads).

Table 8-6 is based on ASTM C789. The waterway area is reduced to account for an assumed 10 inch haunch at each corner. The actual dimension of the haunch may vary slightly among manufacturers.

Table 8-6 Precast Concrete Box Sizes

Span (ft)	Rise (ft)	Waterway Area (ft ²)	Waterway Area (ft ²)
		Single box	Twin boxes
8'	6'	46.6	93.2
8'	7'	54.6	109.2
8'	8'	62.6	125.2
9'	6'	52.6	105.2
9'	7'	61.6	123.2
9'	8'	70.6	141.2
9'	9'	79.6	159.2
10'	6'	58.6	117.2
10'	7'	68.6	137.2
10'	8'	78.6	157.2
10'	9'	88.6	177.2
10'	10'	98.6	197.2
11'	6'	64.6	129.2
11'	8'	86.6	173.2
11'	9'	97.6	195.2
11'	10'	108.6	217.2
11'	11'	119.6	239.2
12'	8'	94.6	189.2
12'	10'	118.6	237.2
12'	12'	142.6	285.2

8.4 Three-Sided Structures and Arches

8.4.1 Design

A three-sided structure or arch may be preferred for those projects where the integrity of the stream bottom must be maintained. These can be concrete rigid frames, concrete arches, or metal arches. Metal arches may be steel or aluminum structural plate.

For all these structures, the footings are designed by the Structural Designer. Refer to Section 5.3 Spread Footings for the design guidelines. For those buried structures on footings, the springing line should be located at or above Q1.1 for ease of construction and longevity.

8.4.2 Metal Arches

The minimum cover is equal to the span divided by 8 or 1'-6" minimum, whichever is greater. The minimum cover should be checked at the face of guardrail from the top of the bituminous pavement. The lateral limit of the soil envelope is 3 feet wide at the footing and extends upward to the subgrade elevation. Backfill requirements should be supplied by the Geotechnical Designer for inclusion in the contract documents.

The plate thickness for arch structures should be specified in the contract documents as shown in the tables in Section 8.4.2.1. If reinforcing ribs are required for the structure, they should be designed by the manufacturer. The thickness of the plates to be used for design should be the thickness stated on the plans minus 0.055 inches or 0.060 inches for aluminum or steel structures, respectively. Refer to Appendix D Standard Notes Structural Plate Structures.

8.4.2.1 Structural Plate Arch Tables

For metal structural plate arches, tables were developed in the same manner as those described in Section 8.2.4 Structural Plate Pipes and Pipe Arches Design Tables Commentary.

Table 8-7 Steel Structural Plate Arches

Span (in)	Min. Cover (ft)	Maximum Fill Height Above Top of Arch (feet) Metal Thickness in inches					
		0.138 in	0.168 in	0.188 in	0.218 in	0.249 in	0.280 in
72	1.5	18	30	30	30	30	30
84	1.5	16	30	30	30	30	30
96	1.5	14	25	30	30	30	30
108	1.5	12	20	25	30	30	30
120	1.5	11	20	25	30	30	30
132	1.5	9	18	20	25	30	30
144	1.5	8	16	20	25	30	30
156	1.6	7	16	18	20	25	30
168	1.8	6	14	16	20	25	25
180	1.9	6	12	16	20	20	25
192	2.0	5	12	14	18	20	25
204	2.1		11	12	16	20	20
216	2.3		9	11	14	18	20
228	2.4		8	10	12	16	18
240	2.5			8	11	14	16
252	2.6			7	9	12	14
264	2.8				8	10	12
276	2.9				7	9	10
288	3.0					7	9
300	3.1					6	8

Table 8-8 Aluminum Structural Plate Arches

Span (in)	Min. Cover (ft)	Maximum Fill Height Above Top of Arch (feet) Metal Thickness in inches				
		0.150 in	0.175 in	0.200 in	0.225 in	0.250 in
60	1.5	25	30	30	30	30
72	1.5	20	25	30	30	30
84	1.5	18	20	25	30	30
96	1.5	16	20	20	25	30
108	1.5	14	18	20	25	25
120	1.5	12	16	18	20	25
132	1.5	11	14	16	20	20
144	1.5	9	12	16	18	20
156	1.6	9	11	14	16	20
168	1.8	8	10	14	16	18
180	1.9	7	9	12	14	16
192	2.0	6	8	10	12	14
204	2.1	5	6	8	10	12
216	2.3	4	5	7	9	10
228	2.4		4	6	7	8
240	2.5		3	5	6	7
252	2.6			4	5	6
264	2.8			3	4	5
276	2.9				3	4

8.4.2.2 End Treatment

A. Steel Structural Plate Arches

The end treatment should be the same as required for steel structural plate pipes and pipe arches. In addition, a top step cut should be provided with a height of about 12 inches.

B. Aluminum Structural Plate Arches

The end skew and bevel, bottom step cut, and end reinforcement should be the same as required for aluminum structural plate pipes and pipe arches, except the top step cut should be increased to about 12 inches whenever it is required.

8.4.3 *Concrete Rigid Frames and Arches*

These structures consist of both proprietary and non-proprietary systems. There is currently no MaineDOT approved list of proprietary systems. Refer to the buried structures technical resource people for the list of acceptable systems.

Precast concrete boxes are detailed on contract plans with only the basic layout and required hydraulic opening, so the Contractor can choose among available proprietary products. The manufacturer is responsible for the design of the structure, including determination of wall thickness and reinforcement. The loading specified for the structure should be HS25 and Special Provision 534 must be included in the PS&E package. If wingwalls are required, refer to Section 5.6.5 Prefabricated Proprietary Walls. In general, a concrete modular wall system is preferred for more extensive walls due to increased longevity.

Spread footing loads should consider all reactions transferred to the footings through the arch walls. A minimum backfill compaction to prevent roadway settlement adjacent to the structure should be provided by the Geotechnical Designer for inclusion in the contract documents. A higher backfill compaction density may be required on structures requiring resistance to large horizontal reactions at the base of the arch wall.

The minimum cover is 6 inches. Whenever possible, the structure should be buried deep enough so that no special treatment is needed for the guardrail posts.

References

AASHTO, 1977 and Interims, *Standard Specifications for Highway Bridges*, Washington, DC

Chapter 9

TIMBER/ENGINEERED WOOD COMPOSITES



West Branch Bridge, Byron



Grist Mill Bridge, Lebanon

CHAPTER 9 – TIMBER/ENGINEERED WOOD COMPOSITES

9	TIMBER/ENGINEERED WOOD COMPOSITES	9-1
9.1	General.....	9-1
9.2	Design	9-1
9.3	Beams	9-1
9.3.1	Glue-Laminated Beams.....	9-1
9.3.2	FRP-Reinforced Glulam Beams	9-2
9.4	Decks.....	9-2
9.4.1	Direct-Span	9-2
9.4.2	Transverse Decking.....	9-2
9.5	Preservative Treatments.....	9-2
9.6	Bridge Rail	9-3
	References	9-4

9 TIMBER/ENGINEERED WOOD COMPOSITES

9.1 General

Timber bridge structures and components should sometimes be considered in the preliminary design of a project. Timber structures may be a viable solution when aesthetics are a driving force in the design. Durability, maintenance, and initial cost are three main factors to consider when evaluating timber alternatives. Skews should be avoided on timber bridge structures due to fabrication and constructability issues. Timber substructures are not recommended. Typically, timber bridges have a much higher initial cost compared to steel or concrete structures.

9.2 Design

Timber beams and other structural members should be designed according to the AASHTO LRFD Specification. Members should be designed for net dimensions of anticipated use conditions. Initial design strength and stiffness values are given in AASHTO LRFD, which are modified for individual use and conditions.

Adequate bracing of all structural members should be used to prevent lateral and rotational deformation. Steel bracing diaphragms are recommended over timber blocks.

Ritter (1992) is a valuable resource when designing timber structures. The book is found in the MaineDOT Library.

9.3 Beams

9.3.1 *Glue-Laminated Beams*

Glue-Laminated (glulam) beams can be considered as an aesthetic alternative for short span bridges. Single span glulam timber bridge structures are feasible up to about 50 feet. Initial bending strengths should be modified to take into account volume effects. Beams should be cambered a minimum two times the unfactored dead load deflection.

Glulam beams may be constructed of most wood species, assuming they conform to AASHTO M168 and can be treated for preservation. Costs should be determined during the preliminary design phase before deciding to use a species outside those defined in the code. The beam properties are calculated from the individual members that make up the glulam. The University of Maine has a computer program that can determine the beam properties given the properties of the components.

9.3.2 FRP-Reinforced Glulam Beams

Glulam beams can be used on longer spans when reinforced with fiber-reinforced polymers (FRPs). The beam depth can also be reduced with FRPs.

Direct involvement with the University of Maine is required for any project utilizing FRPs. The University has the resources and lab equipment to predict the beam properties and verify the values for design.

Commentary: The design of structures utilizing FRP reinforcement of timber structures is currently not addressed in the AASHTO bridge design codes.

9.4 Decks

9.4.1 Direct-Span

Glulam decks and transversely post-tensioned decks are two options for short span structures. Direct-span decks are feasible up to about 33 feet.

9.4.2 Transverse Decking

There are two types of timber decking: solid sawn lumber and glulam.

Solid sawn lumber should be used only on low volume roads. No less than a 4x4 member should be used for the deck size. Timber running boards nailed to the deck are a common wearing surface for this deck type.

Glulam decks may be used for wider beam spacings. Glulam decks are the preferred option with glulam beams. A double layer of waterproofing membrane with a bituminous wearing surface should be used to protect the deck. Glulam decks are typically placed at a constant cross slope across the bridge. Positive drainage of the wearing surface should be achieved by super elevating the deck or by constructing cross slope in the wearing surface during the final paving. A retainer angle should be placed along the curb line to protect the pavement edge.

9.5 Preservative Treatments

All timber members must be pressure-treated to prevent microorganism and insect damage. Treated members are not allowed to be in contact with the stream or river. Creosote and pentachlorophenol are treatments that have been used in past projects. The Structural Designer must research the required treatment to match the wood species and application. The Structural Designer must also coordinate the treatment with the Environmental Coordinator. The specification for the treatment must meet the latest American Wood-Preservers Association standards.

Where possible, all holes should be predrilled into the members before a preservative treatment is applied. If field drilling is required, the holes should be treated with an approved treatment.

9.6 Bridge Rail

Timber bridge rail is available up to a crash-test rating of TL-4. Refer to Section 4.4 Bridge Rail. Steel-backed timber guardrail can be used on the approaches when it is appropriate.

References

American Wood-Preservers Association, 1995, *American Wood-Preservers Association Book of Standards*

AASHTO, 1998 and Interims, *Load and Resistance Factor Design (LRFD) Bridge Design Specifications*, Washington, DC

Ritter, Michael A, 1992, *Timber Bridges Design, Construction, Inspection, and Maintenance*, USDA Forest Service, Washington, DC

Chapter 10

REHABILITATION



Pope Memorial Bridge, East Machias



Waldo-Hancock Bridge, Prospect-Verona

10	REHABILITATION.....	10-1
10.1	General.....	10-1
10.2	Superstructure Rehabilitation	10-1
10.2.1	Evaluation.....	10-1
10.2.2	Wearing Surface Replacement/Rehabilitation	10-2
10.2.2.1	General	10-2
10.2.2.2	Bituminous Wearing Surfaces	10-3
10.2.2.3	Concrete Wearing Surfaces	10-4
10.2.3	Deck Replacement/Rehabilitation	10-4
10.2.4	Superstructure Replacement/Rehabilitation	10-4
10.2.5	Evaluation of Deck Cores	10-4
10.2.6	Bridge Widening	10-5
10.3	Bearings	10-5
10.4	Expansion Devices	10-6
10.5	Bridge Rail and Connections	10-7
10.5.1	General.....	10-7
10.5.2	Retrofit Policy	10-8
10.5.2.1	Interstate System	10-8
10.5.2.2	Non-Interstate System.....	10-8
10.5.2.3	Existing Bridges on Highway Projects	10-8
10.6	Substructure Rehabilitation.....	10-12
10.7	Substructure Reuse	10-13
10.7.1	General.....	10-13
10.7.2	Timber Pile Foundations.....	10-13
10.7.3	Granite or Stone Substructure	10-14
10.8	Major Rehabilitation Strategy.....	10-15
10.9	Seismic Retrofit	10-16
10.9.1	General.....	10-16
10.9.2	Criteria for Evaluation	10-16
10.9.3	Analysis	10-17
10.9.4	Scheduling of Seismic Retrofit Work	10-18
10.9.5	Retrofit Costs.....	10-18
10.9.6	Procedure.....	10-19
10.9.7	Seismic Retrofit Systems.....	10-19
10.10	Buried Structures	10-21
10.10.1	Invert Lining	10-21
10.10.2	Sliplining	10-22
	References	10-24

Table 10-1	Bridge Joint Modification Types	10-7
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Figure 10-1 Interstate Rails Attachment Type "A" – Rail Types B, C, E, & F. 10-10

Figure 10-2 Interstate Rails Attachment Type "A" – Rail Types G, H, J, & K. 10-11

Figure 10-3 Interstate Rails Attachment Type "A" - Rail Types L, M, & Z 10-12

Figure 10-4 Non-NHS System Rail - Attachment Type "B" - Rail Retrofit 10-12

10 REHABILITATION

10.1 General

Bridge rehabilitation involves structural or functional upgrades to an existing bridge that leave part of that bridge remaining in place. The extent of the rehabilitation can range from relatively minor work such as replacement of curb and rail, to major work such as replacement of all deficient concrete in a multispan arch structure. Usually, a preliminary scope is defined that outlines the expected rehabilitation work. The Designer will review this scope, and through preliminary design analysis, determine whether the expected work is the optimal course for improvements to the bridge. It is very important that the contract clearly describe the expected repairs and the extent of work to avoid excessive and costly change orders in the field.

Rehabilitation work can be classified as minor or major rehabilitation. Minor rehabilitation addresses non-structural repair such as concrete surface repair, deck overlays, joint and bearing restoration, steel secondary member repair, and minor repair to primary steel members. Major rehabilitation involves structural repair or replacement of primary bridge elements, and includes such work as pier cap or pier replacement, deck replacement, superstructure replacement, bridge widening, and primary member replacement or strengthening.

Most rehabilitation projects should have a life cycle analysis done to confirm that rehabilitation is preferred over replacement. Refer to Section 2.2 Economic Comparisons for more information.

A scour evaluation should be completed for all structures for which the scope of rehabilitation exceeds deck, wearing surface, or rail rehabilitation/replacement. If the structure is scour critical, the appropriate counter measure should be investigated as part of the rehabilitation project.

10.2 Superstructure Rehabilitation

10.2.1 Evaluation

The most common superstructure rehabilitation projects involve, in order of complexity, wearing surface replacement, deck replacement, or superstructure replacement. The degree of work is dependent upon the condition of the existing structure, which must be evaluated during preliminary design.

The Designer will compile the data needed for this evaluation. Activities include review of existing plans to determine rebar cover, slab thickness, type of original wearing surface, and presence or absence of membrane. Inspection and maintenance work reports are reviewed for the wearing

surface, deck, and superstructure condition rating and description, and any maintenance work that has been required. The substructure condition rating should be noted, and an estimated remaining life of the substructure determined. Discussions with Bridge Maintenance will be useful to compare the predicted substructure service life to the expected life of the repair.

Field inspection should be done to document leakage and efflorescence, potholes, cracks, delamination, and spalling of the deck, as well as the condition of deck joints, curbs, and railing. Deck cores may be obtained at the discretion of the Designer to provide representative sampling for testing and to document the condition of the deck. Typical locations for deck cores are at the curb line and the center of wheel paths. Refer to Section 10.2.5 Evaluation of Deck Cores for a discussion of deck core interpretation. The need for concrete cores should be determined at the project kick off meeting and coordinated with the Project Manager. Refer to the Getting Started Chapter of the Project Management Guide for guidance.

Maintaining traffic during construction can cause issues in these projects. Refer to Section 2.4 Maintenance of Traffic During Construction.

10.2.2 Wearing Surface Replacement/Rehabilitation

10.2.2.1 General

This work involves replacing the existing wearing surface with a new partial or full-depth wearing surface. Material used can be either concrete or bituminous. A concrete wearing surface should be used only for those cases noted in Section 4.6 Wearing Surfaces. For all other wearing surface replacements, with or without rehabilitation of the existing deck, replace with 1/4" membrane, and 3 inches of hot bituminous pavement.

Selected areas of the deck may need to be repaired as discussed in Section 10.2.3 Deck Replacement/Rehabilitation. The removal is described as extending to rebar or extending below rebar, depending upon how extensive the deterioration. For minor deck rehabilitation, up to 5% of the existing deck area is removed below rebar, and up to 15% is removed to rebar. For most wearing surface projects, items for deck rehabilitation should be included in the contract, due to uncertainty of actual field conditions. Typical items for deck rehabilitation are described in Standard Specification Section 518 – Structural Concrete Repair and are as follows:

- Item 518.50 Repair of Upward Facing Surfaces to Reinforcing Steel, < 7.9 inches
- Item 518.51 Repair of Upward Facing Surfaces below Reinforcing Steel, < 7.9 inches

- Item 518.52 Repair of Upward Facing Surfaces, ≥ 7.9 inches

If the existing deck slab is expected to have a rough and irregular surface that could puncture the membrane waterproofing, the Designer should specify high performance membrane. A high performance membrane should also be considered if there are issues with vehicles breaking at the bridge, the performance of the previous membrane, or the design life of the rehabilitation project. Refer to Section 4.7 Membranes for further guidance. A rough surface may be expected on a deck that is to be scarified or where a well-bonded concrete wearing surface is to be removed.

Commentary: The Resident also has the option of modifying the specified membrane in the field, depending upon the roughness of the concrete surface. The Resident may choose to add a second layer of standard membrane when high performance membrane was not specified. In cases of extremely rough surfaces, the Resident also has the option of placing bituminous shim directly on the concrete prior to installing membrane, or substituting the bituminous and membrane with a concrete wearing surface. Both of these materials will be obtained through change order procedures and should not be estimated or shown on the plans.

If the condition of the railing, curb, or joints is substandard, replace or modify to current standards. Refer to Sections 4.4 Bridge Rail and 10.5 Bridge Rail and Connections for further guidance.

10.2.2.2 Bituminous Wearing Surfaces

Partial depth replacement of a bituminous wearing surface is known as a wearing surface rehabilitation, and is indicated if the deck condition is good, and there is an effective existing membrane. Rehabilitation may also be indicated as a low cost measure to prolong the life of a poor quality deck.

Full depth replacement of the wearing surface should be done when the deck is in good condition, but no membrane is present.

For bituminous wearing surface replacement of less than 3000 ft² of deck area, the surface should be prepared by sandblasting, or by using a scabber. For deck areas greater than 3000 ft², the cost of scarifying equipment can be justified; specify scarifying the deck at least 1/2" when the chloride content is low, and 3/4" if the chloride content is high.

Certain situations may warrant a modification of the above or a different solution to provide an adequate wearing surface in order to meet depth, crown, or other existing conditions.

10.2.2.3 Concrete Wearing Surfaces

If a concrete wearing surface is to be placed, a good bond with the deck is essential to prevent future maintenance problems. Scarify or scabble the deck, and then blast before applying the new wearing surface. Where non-integral concrete wearing surfaces are used, a 2 inch minimum unreinforced concrete wearing surface should be specified. Depths of unreinforced concrete should not exceed 4 inches.

10.2.3 Deck Replacement/Rehabilitation

Rehabilitation of a deck involves removing selected areas of concrete down to sound concrete, and replacing with new concrete. Major deck rehabilitation is classified as removal of concrete below rebar for 5% to 40% of deck area and removal to rebar for greater than 15% of deck area. For deck areas greater than 3000 ft², scarify the existing deck. If the condition of the railing and curb is substandard, replace or modify to current standards.

In general, deck replacement should be performed if more than 40% of deck area is deficient below rebar. If more than 30% of deck area is deficient below rebar, a life cycle cost analysis should determine whether deck rehabilitation or replacement is warranted. Refer to Section 2.2 Economic Comparisons for more information.

10.2.4 Superstructure Replacement/Rehabilitation

A life cycle analysis described in Section 2.2 Economic Comparisons may show that superstructure replacement is less costly than deck replacement, especially if the existing superstructure consists of painted steel girders. This is because the cost of painting steel often exceeds that of new steel due to paint containment costs.

Superstructure replacement may require substructure modifications, such as placement of a reinforced concrete cap to adequately distribute the loads.

10.2.5 Evaluation of Deck Cores

The purpose of testing existing decks is to assist the Designer in judging the extent of deck rehabilitation or replacement that is warranted. The testing should include chloride content and compressive strength, and may also include rebar inspection and shear strength.

Chloride content is sampled in the top 1/2", and then every inch thereafter. If the level is below 1.35 lb/yd³, it is considered to be in a "non-corrosive atmosphere." The depth of concrete that should be removed can be estimated

based upon the depth where corrosivity diminishes below 1.35 lbs/yd³ or solid concrete is found.

Compressive strength is sampled below the top 1/2" at the depth where the concrete can be cored intact. When a core cannot be taken effectively, the concrete should probably be removed. Core compressive strength should be compared to the expected design strength, and a judgment made by the Designer as to the extent of rehabilitation or the need for replacement.

If a core sample happens to go through rebar, the depth of steel is noted, and a visual inspection notes the degree of corrosion. Chloride content above and below the rebar may be taken.

Occasionally, a shear test between the existing asphalt and concrete will be done to determine the potential bond between the new surfaces. Values vary widely, from as low as 50 psi to as high as 1000 psi. These values may be used to determine the level of effort required to remove the existing wearing surface. Good engineering judgment should be used when interpreting the shear test results.

10.2.6 Bridge Widening

Widening an existing structure to meet current standards may be cost effective if the condition of the existing substructure is good. Usually the structure should be widened to only one side, for ease of construction. The widened superstructure will be supported either on a widened substructure, or may be cantilevered from the existing substructure. An analysis of the capacity of the substructure by the Geotechnical Designer will determine whether a cantilever is feasible.

If a deck slab overhang is increased without adding girders, the existing exterior girder must be analyzed with the additional load, both during concrete placement and in final position. A torsional analysis will usually be required.

10.3 Bearings

When a bridge is to be rehabilitated, the bearings should be evaluated for the need to repair or replace them. Depending upon the expected life of the structure, repairing the existing bearings may be preferred over replacing with modern bearings. In some cases, no repair at all will be the most cost effective and practical solution.

Many existing steel bridges have rocker bearings that can be removed, refurbished, and then replaced. Contact Bridge Maintenance for further guidance on the rehabilitation of existing rocker bearings. If the bridge is in an SPC B seismic area, rocker bearings should be replaced with elastomeric or other

bearing systems, as discussed in Section 10.9 Seismic Retrofit. A widened structure should be fitted with the same bearing type as that installed on the remaining structure for each substructure unit.

10.4 Expansion Devices

On a wearing surface replacement or deck rehabilitation project, the bridge expansion devices (joints) should be examined to determine their condition. The joint armor may be damaged, or the seal may be gone. The value of replacing the seal, repairing the joint armor, or replacing the entire joint should be assessed for each project. The Designer must consider the potential damage to the structure below if repairs or modifications are not made, as well as the expected life of the structure before full bridge replacement is warranted.

Often the joint must be modified or raised to accommodate the increase in grade created by additional pavement. If the joint armor is not damaged beyond repair, and a compression seal can be used, the joint should be modified by welding a round bar to the top of the joint armor. If the joint armor is damaged, the affected steel can be cut out and replaced with a new piece. Keeper bars should be added to the joint armor if not part of the existing joint configuration.

To select a new seal, field measurements must be taken to determine which manufacturer's seal will fit. The existing joint opening should be measured, along with the temperature and the location of the keeper bars if applicable. With this information, the maximum and minimum expected joint opening can be determined. The Designer should then use the manufacturer's literature from the two suppliers listed in Table 4-7 to determine the minimum installation opening and seal depth. A seal can be selected to fit within the given parameters (depth of seal, minimum installation opening, and movement rating) by using Table 4-7 Elastomeric Joint Seal Movement Ratings or the Bridge Compression Seals for Expansion Joints section of the Qualified Products List. The depth from top of new joint to top of seal should comply as closely as possible with the Standard Detail 520(10) minimum of 1/2".

For bridges with differential movement, excessive rotation at the joint, or if the joint space is measured and found to be uneven from one side of the bridge to the other, a gland seal may be selected instead of a compression seal.

In some cases, the existing seal type may be changed without modification of the existing joint armor. Prequalified seals listed in Section 4.8 Deck Joints and Expansion Devices should be evaluated for use inside existing joint armor.

If a prefabricated seal cannot be found to fit the existing joint armor, self-leveling joints can be considered. For the approved list of self-leveling joints refer to the Bridge Expansion Joint Systems section of the Qualified Products List on the

MaineDOT website. These seals are a temporary solution, with a service life of only six to seven years.

Modifications and replacement of existing joints should be specified in accordance with Table 10-1. The descriptions of these joint modifications are not meant to be all-inclusive but merely a broad description. The Designer should use good judgment in determining which type of modification to specify. As a general rule, only the pay items listed here should be used. When two joint modifications required on the same project must be detailed separately, or may have a significant difference in cost, but fall under the same type, use the same pay item number for both modifications and change the pay item description by adding letters to differentiate (i.e. Type 3A and Type 3B would both be paid under pay item 520.243). Construction requirements and modification type descriptions are specified in Special Provision Section 520 Expansion Devices. The Designer must verify that the PS&E package contains this Special Provision.

Table 10-1 Bridge Joint Modification Types

Item Number	Modification	Scope of Work	Examples of Work Scope
520.241	Type 1	Normal maintenance	<ul style="list-style-type: none"> • Replace seal* • New keeper bars
520.242	Type 2	Steel repair	<ul style="list-style-type: none"> • Repair joint armor steel • Minor concrete repair
520.243	Type 3	Concrete repair	<ul style="list-style-type: none"> • Concrete removal and repair
520.244	Type 4	Modification	<ul style="list-style-type: none"> • Modification to new joint type
520.245	Type 5	Replacement	<ul style="list-style-type: none"> • Full joint removal and replacement

* Seal replacement is assumed on all other Bridge Joint Modification Types

10.5 Bridge Rail and Connections

10.5.1 General

Bridge rehabilitation projects and resurfacing projects should consider the need for the replacement, retrofitting, or retention of existing bridge rails. In general, bridge rails should be replaced or retrofitted to meet AASHTO LRFD standards. Refer to Section 4.4 Bridge Rail for further guidance.

For rehabilitations where it is desirable to leave the existing end posts in place and the bridge transition is in question, it is acceptable to use Bridge Transition Type 2 as shown in Standard Detail 606(26).

10.5.2 Retrofit Policy

10.5.2.1 Interstate System

Bridge rails on the interstate system have been identified as shown in Figure 10-1, Figure 10-2, and Figure 10-3. The policy for retention, replacement, or retrofit for these existing bridge railings on the interstate system is as follows:

- Type B, C, E, H, & K: Either replace the existing rail and curb system with F-shaped barrier or retrofit existing rail and curb system with a crash-tested retrofit system.
- Type F, G, J, & L: Retain existing rail and curb system. Consider replacing rusted toggle bolts on Type J.
- Type M & Z: Retain existing rail and curb system. Retrofit splice detail.

Bridge rails similar to the above interstate bridge rails on non-interstate systems should be treated similarly as prescribed for the interstate system except as otherwise discussed here.

10.5.2.2 Non-Interstate System

Retention of existing sound substandard bridge railings is acceptable on non-interstate systems for economic reasons when the bridge has a low accident history (CRF < 1.0), and has either a low posted speed limit (mph < 45), or a low traffic volume (AADT < 400).

Retrofitting of existing substandard bridge railings on non-interstate systems having sound concrete posts should be considered utilizing 10 gauge thrie beam with block-outs on posts not exceeding a spacing of 10'-6" (refer to Figure 10-4). The thrie beam must be specified as 10 gauge on the plans since the Standard Specifications call for the thinner 12 gauge. Top of thrie beam should be 2'-10" above traveled way and curb offset should not exceed 3-1/2". Existing substandard railings behind the thrie beam should remain in place. This retrofit is based on a Michigan crash tested retrofit.

10.5.2.3 Existing Bridges on Highway Projects

For a bridge within the limits of an NHS Arterial Program project, the existing bridge rail should be considered for replacement, retrofitting, or retention as part of the highway project. The only exception to this is when

the bridge has been scheduled for additional work as a separate Bridge Program project.

For a bridge within the limits of a non-NHS highway project that is not otherwise programmed for work, the existing bridge rail does not require consideration for improvements as part of the highway project. However, a rigid guardrail to bridge connection and additional stiffening posts in the approach rail should be provided.

For a bridge just outside the project limits of a highway project, the existing bridge rail need not be considered for improvements. However, if the approach guardrail is within NHS highway project limits, then the bridge connection should be upgraded to current standards. Non-NHS projects should have a rigid guardrail to bridge connection and additional stiffening posts in the approach rail provided within the highway project limits.

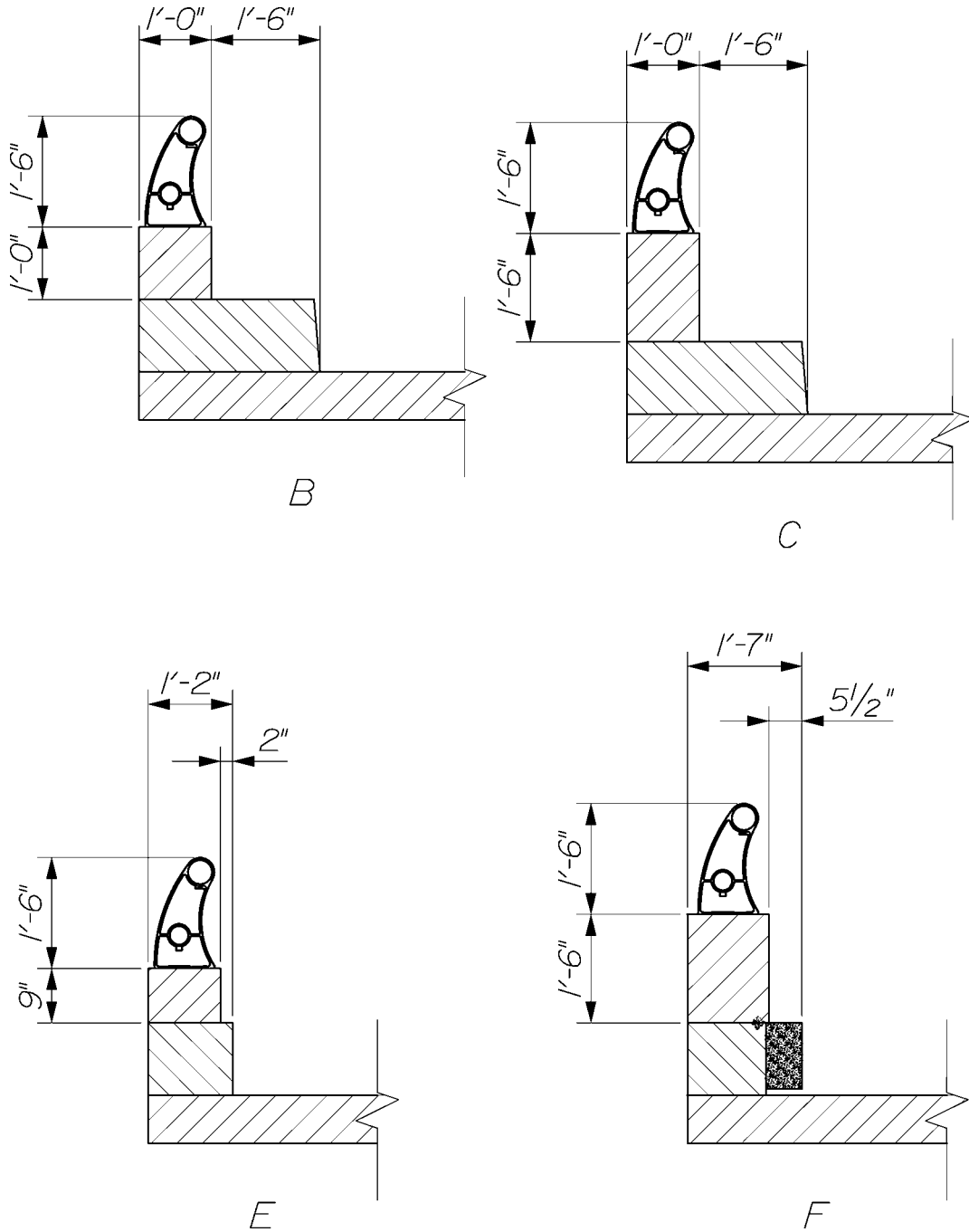


Figure 10-1 Interstate Rails Attachment Type "A" – Rail Types B, C, E, & F

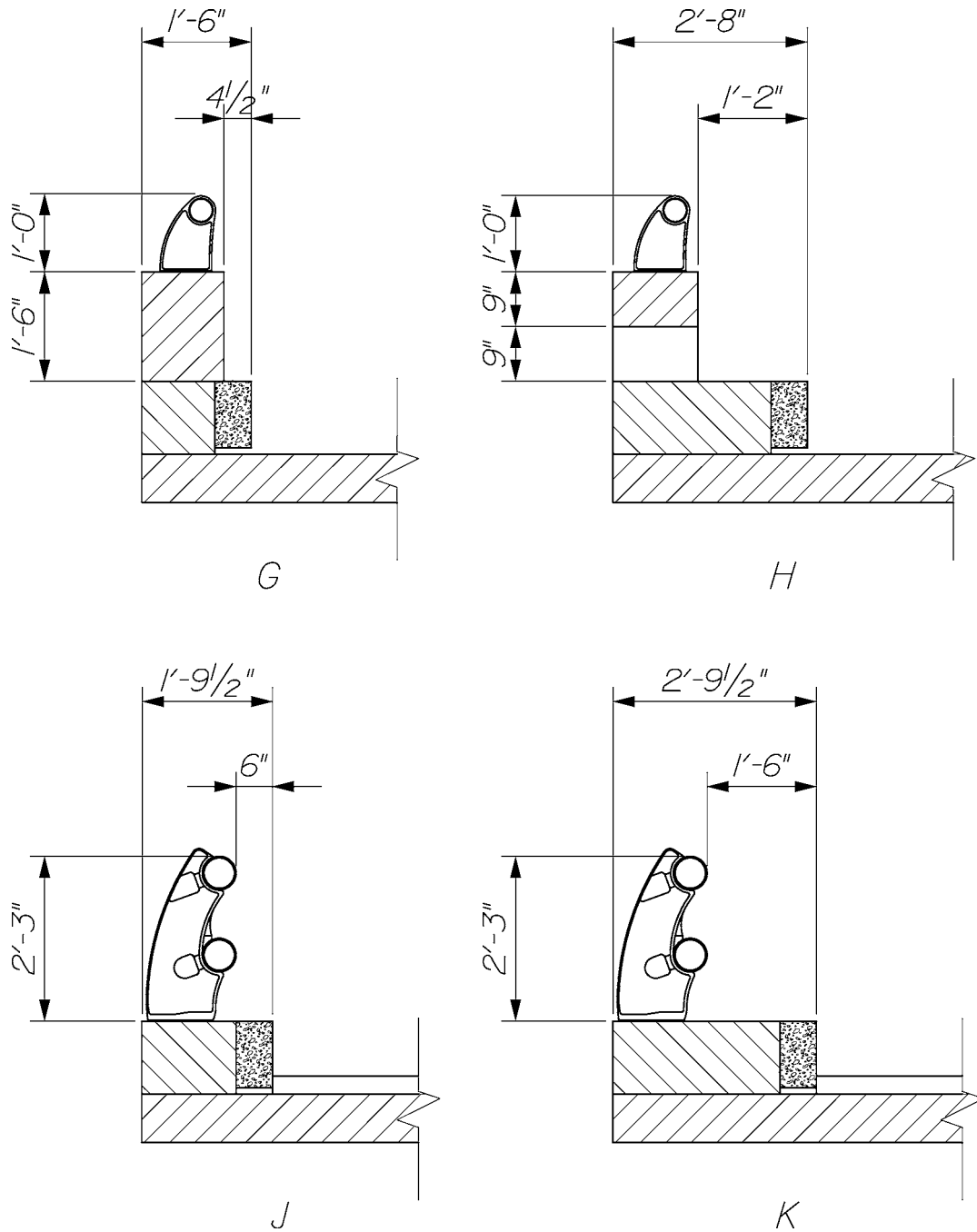


Figure 10-2 Interstate Rails Attachment Type "A" – Rail Types G, H, J, & K

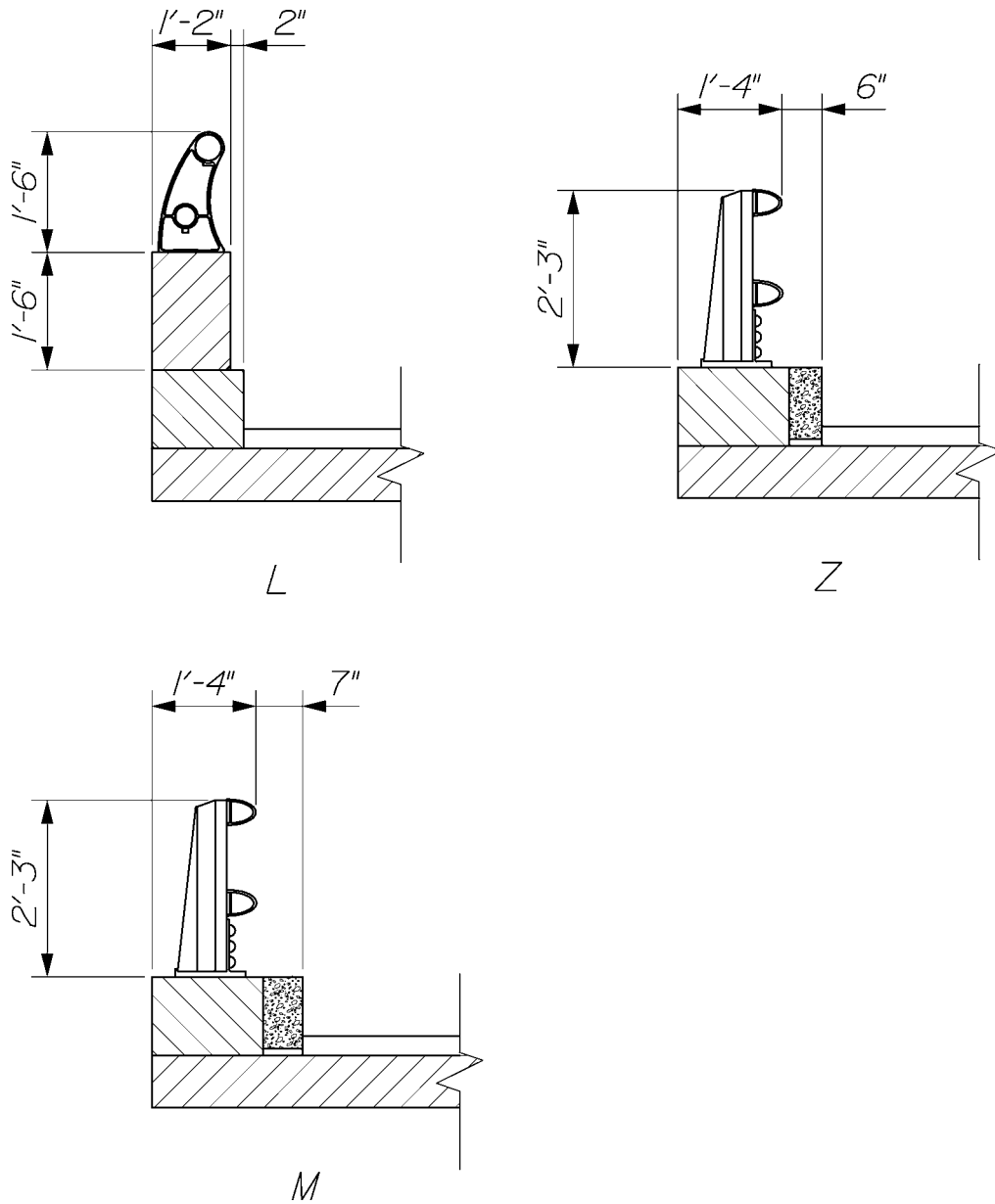


Figure 10-3 Interstate Rails Attachment Type "A" - Rail Types L, M, & Z

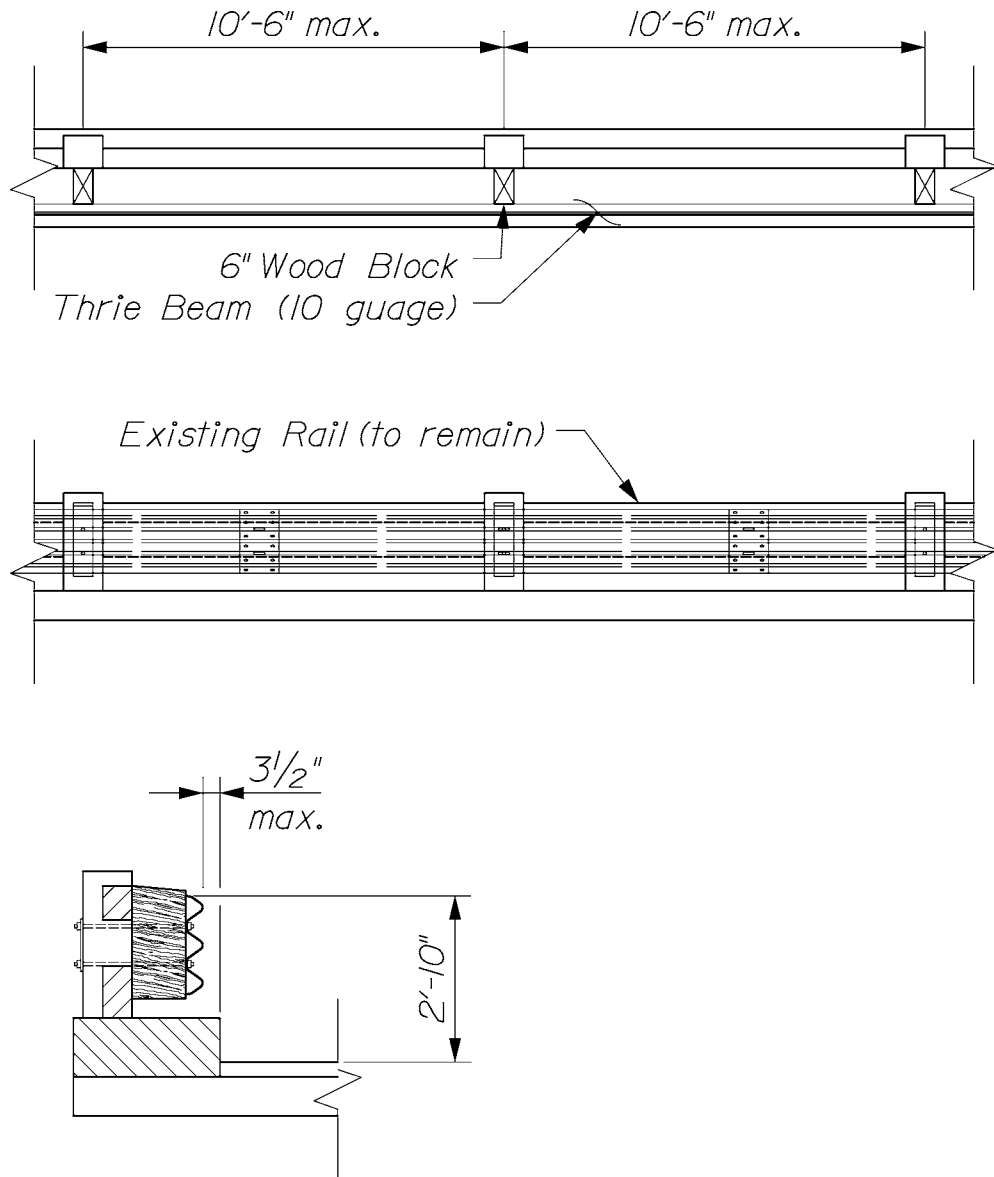


Figure 10-4 Non-NHS System Rail - Attachment Type "B" - Rail Retrofit

10.6 Substructure Rehabilitation

Substructure rehabilitation work may involve such activities as scour repair, jacketing an abutment or pier, grouting of a granite block abutment or pier, or post-tensioning an unreinforced pier cap.

Where substructures have rotated due to inadequate bearing capacity, the angular distortion due to rotation can be restored in some cases with the use of underpinning. Where substructures are actively rotating, underpinning can be used to stop or decrease the magnitude of the movement. Underpinning consists of increasing the foundation soil bearing capacity by either driving

structural elements (piles) or constructing cast-in-place elements (micropiles) adjacent to or under the existing foundation. In some cases, the footing dimensions need to be extended to incorporate the underpinning elements. Underpinning can also be used to increase foundation capacity for substructures that are to remain in place when the superstructure dead and live loads are increased, as with a superstructure widening. The feasibility of the use of underpinning for substructure rehabilitation should be evaluated by the Geotechnical Designer.

10.7 Substructure Reuse

10.7.1 General

When an existing substructure is to be reused with new loads applied, the existing foundation should be evaluated to assure adequate capacity. When not known, determination of the existing foundation geometry and condition should be made through exploration and testing. Where foundation deterioration is suspected or indicated, such as pile section loss or weakening due to corrosion or decay, a structural analysis should be conducted to evaluate the effects of the deterioration.

10.7.2 Timber Pile Foundations

Where an existing timber foundation is being considered for reuse, the condition of the existing timber piles should be assessed and the capacity of the piles evaluated. The evaluation for reuse needs to be appropriate to the particular site. FHWA estimates a 50-year life span for timber piles in a marine environment. A typical procedure for timber pile investigation and evaluation should include any appropriate combination of the following:

- o Obtain cores of at least one pile from each foundation to evaluate soundness of the pile and the presence of marine borers if applicable.
- o Conduct at least one static pile load test to 2.5 times the proposed pile design load.
- o Conduct at least one pile integrity test (impact echo test) to evaluate the structural integrity of piles and estimate the length of the piles.
- o Evaluate groundwater conditions and subsurface conditions with borings.
- o Assess the theoretical capacity of the piles using confirmed soil statigraphy.

10.7.3 Granite or Stone Substructure

If “as-built” plans cannot be found, an investigation to determine the granite or stone abutment configuration should be performed. If “as-built” plans are available, efforts should be directed toward verifying their correctness. The abutment investigation strategy chosen by the Geotechnical Designer needs to be appropriate to the particular site.

A typical procedure for preliminary abutment investigation includes the following:

- o Obtain existing records such as “as-built” plans, etc.
- o Assess the condition of the existing substructures. Document:
 1. indications of foundation instability (settlement, sliding, or overturning), deterioration of materials (pointing mortar, stones)
 2. localized bulging, rotation of stones
 3. location of cracks, modifications such as concrete caps or facing, and the condition of the modified portions
 4. drainage issues
- o Conduct a subsurface investigation to verify abutment geometry and integrity as outlined below.
- o Evaluate the reuse potential of the substructure relative to the proposed alignment, width, grade, and loads.
- o Perform a cost analysis to determine whether the reuse or reuse with retrofitting alternative is a cost savings compared to new construction.

In order to verify abutment geometry and integrity, the Geotechnical Designer should conduct the subsurface investigation that is appropriate to the particular site. This investigation may include the following:

- o *Ground Penetrating Radar (GPR) Survey:* A GPR survey is a useful and economical tool that can be used to determine abutment geometry. A geophysicist’s report with an interpretative picture of the geometry of the wall is to be submitted to the Geotechnical Designer.
- o *Borings:* Standard wash borings and augers are taken behind each abutment to verify the geometry of the abutment back and footing, and to acquire data on the backfill and foundation material.

- o *Test pits:* Test pits are dug to confirm footing dimensions, foundation material, and depth.
- o *Seismic techniques:* Various seismic methods, such as crosshole seismic refraction, can be used to define the abutment geometry.

Where reuse of a substructure is selected for final design, stability analyses should be performed as described in Chapter 5, Substructures. The analysis needs to demonstrate that the reused or retrofitted substructure achieves or exceeds the minimum factors of safety for overturning, sliding, and bearing capacity under the proposed grades, widths, and superstructure loads.

10.8 Major Rehabilitation Strategy

Large rehabilitation projects occur on long bridges where replacement costs are high, and a life cycle analysis shows that keeping the existing structure in service is more cost effective than replacement. Another project may involve a historic bridge that is rehabilitated rather than being replaced in an effort to salvage it. Work often consists of repairing parts of the bridge that may be difficult to assess, such as vast areas of concrete or wrapped cables. The work may involve both the superstructure and substructure.

A thorough investigation of the extent of work required is important prior to advertising the project. The investigation should include concrete core samples taken at strategic locations and exposing any hidden components that may be in disrepair. During the final design phase, experienced design, construction, and maintenance team members should spend time in the field physically identifying and marking those areas or members that should be rehabilitated. This information must be transferred to the contract drawings and documents.

Historically, these projects tend to overrun the budgeted cost due to unforeseen conditions. Cost estimates should include adequate contingencies to cover any unexpected findings. Concrete rehabilitation or replacement items that are dissimilar in nature should be paid for under separate pay items.

A mandatory pre-bid meeting should be conducted on site to explain how the proposed repair areas were delineated. At this meeting, one of the proposed repair areas may be removed by a maintenance crew for demonstration purposes. All available test reports, documents, and other data relating to the condition of the bridge should be made available to the bidders. Such information may influence or provide information that may affect the bid process or the construction work effort.

10.9 Seismic Retrofit

10.9.1 General

The Structural Designer should evaluate the seismic failure vulnerability of bridges programmed for rehabilitation. The Structural Designer should then assess options for seismic retrofit measures that will mitigate or eliminate failure vulnerability.

Commentary: Included here are guidelines for determining when seismic retrofit is warranted and what measures should be considered. The retrofit guidelines present concepts in seismic retrofitting, but should not be considered as restricting innovative designs which are consistent with good engineering practice. Much of the guidelines presented are taken directly from the New York Department of Transportation's Interim Seismic Policy.

The primary goal of seismic retrofitting is to minimize the risk of the collapse of all or part of a bridge, and the loss of the use of a vital transportation route, which may pass over or under a bridge. Because of the difficulty and cost associated with strengthening a bridge to current seismic standards, it is not usually economically feasible to do so. For this reason, the goal of seismic retrofitting is limited to preventing unacceptable collapse modes while permitting a considerable amount of structural damage during an earthquake. The unacceptable modes of failure are:

- o Loss of support at the bearings that will result in a partial or total collapse of the bridge
- o Excessive strength degradation of the supporting components
- o Abutment and foundation failures resulting in a loss of accessibility of the bridge

10.9.2 Criteria for Evaluation

Refer to Section 3.7 Seismic for seismic loading criteria. In addition, the following criteria should be considered:

- o *Age and condition of the bridge:* An unusually high seismic vulnerability may justify seismic retrofit or replacement of a bridge with little service life remaining.
- o *Rehabilitation project scope:* The nature and extent of scheduled rehabilitation work can influence the decision to include or defer the recommended seismic retrofit activities.

10.9.3 Analysis

Refer to Section 3.7.2 Seismic Analysis for a discussion of seismic categories listed here.

- o *SPC A bridges:* These bridges in general, will not require seismic retrofit. However, “essential” bridges programmed for major rehabilitation should be considered for seismic retrofit measures described below.

For example, consider replacing tall steel rocker bearings with a more flexible bearing such as an elastomeric bearing if extensive bearing restoration work is already required. Tall rocker bearings may fail in shear and topple. Elastomeric bearings can be used to achieve a more uniform load distribution or direct load to the desired substructure. Merely by adjusting the height and shear stiffness of the elastomeric bearing, the distribution of seismic forces can be controlled.

Another retrofit measure is to replace the existing bearings with more sophisticated energy dissipating devices. These dissipaters limit the seismic force to the superstructure, thereby limiting the damage to the substructure.

Continuity is also a concern. A multi-simple span bridge does not have the same degree of redundancy as a continuous bridge. Consider providing a continuity retrofit at piers supporting simple spans if bearing or deck replacement work is contemplated. Cable restraints should be considered at piers where the available support length is inadequate and a continuity retrofit is not being considered.

- o *SPC B bridges:* The recommended retrofit actions are as follows:

Replace tall rocker bearings with a more flexible bearing type or an energy dissipating device, and replace or retrofit companion fixed bearings.

Replace short steel sliding bearings (6 inches deep or less) on “essential” structures and on structures requiring bearing restoration work. Short steel fixed bearings that are in good condition may be kept or replaced as conditions warrant.

Exceptions may be made to this general guideline when bridges are extremely wide with many stringers in cross section, or when continuous over several supports and bearings are functioning properly and in good condition. Fixed or tall steel expansion bearings supporting non-redundant elements should always be replaced.

Provide continuity at piers for multiple simple span bridges. When conditions permit, the preferred method is to retrofit beams at piers by splicing for continuity. Where this is not feasible, cable restrainers or other connecting devices should be added.

Widen bridge seats where appropriate.

Add lateral restraint systems at substructures.

Concrete columns should be evaluated for reinforcement details. In general, it is preferable to use the principles of seismic isolation by upgrading the bearings rather than retrofitting substructure components. Certain types of bearings can alter the dynamic response of a bridge and as a consequence, can reduce superstructure forces by a factor of 5 to 10.

- o *Bridges with special conditions:* Consider additional retrofit measures or structure replacement for bridges with vulnerable components discussed in Section 3.7.1.5 Structure Type and Detail, especially if functionally important, and especially if located in an SPC B area.

10.9.4 Scheduling of Seismic Retrofit Work

For "essential" bridges and bridges with special conditions, seismic retrofit work should be included in the first scheduled general rehabilitation activity for the structure.

For other SPC B bridges, the seismic retrofit work should be included in the next scheduled major rehabilitation work. Minor rehabilitation contracts should include as much of the seismic retrofit work as can be accommodated by the project cost and compatibility of activities. At a minimum, cable restraints or continuity at piers should be included where necessary, and lateral restraint systems should also be included. Bridges with tall steel rocker bearings should be scheduled for follow-up retrofit activity, if necessary.

10.9.5 Retrofit Costs

The cost of retrofitting structures will vary significantly based on the type and extent of needed work, as well as site conditions. It may be appropriate in some cases to limit immediate retrofit action to a predetermined cost ceiling, while deferring remaining less critical actions to a future project. As a guideline, a cost increase in the range of 10% -15% is considered appropriate, with 15% being typical when project cost is less than \$2 million. When a bridge is considered to be highly vulnerable, a cost increase in excess of 15% may be warranted to guarantee the structural integrity of the bridge.

In general, a Structural Designer's decision to defer seismic retrofit work for SPC B bridges should be made in concurrence with the Engineer of Design, with appropriate documentation made.

10.9.6 Procedure

According to the map in Figure 3-4, only bridges in the extreme northwest of the state are classified as SPC B, with the remaining bridges classified as SPC A. According to the AASHTO Standard Specifications, no detailed seismic analysis is required for any bridge in SPC A or for any single span bridge. However, the connections must be analyzed for specified static forces, and the supports must meet the given minimum support lengths. In addition to these requirements, "essential" bridges in SPC A should be evaluated based on the procedures outlined for SPC B.

The detailed seismic evaluation of a bridge in SPC B should be performed in two phases. The first phase is a qualitative analysis of individual bridge components using one of the methods described in Section 3.7.2 Seismic Analysis. Once the analysis is performed, and the resulting forces and displacements (referred to as demands) are determined, they are compared with the ultimate force and displacement capacities of each of the components. A capacity/demand (C/D) ratio is then calculated for each potential mode of failure in each component. The ratio denotes the portion of the design earthquake that each of the components is capable of resisting.

The second phase of evaluation is an assessment of the consequences of failure in each of the components. Consideration should be given to retrofitting substandard components if their failure results in bridge collapse or, in some cases such as "essential" bridges, the loss of function. A flow chart detailing this procedure is shown and discussed in FHWA, May 1995.

10.9.7 Seismic Retrofit Systems

Seismic retrofit systems are designed to prevent collapse and/or severe structural damage of the bridge due to the following modes of failure:

- o Bearing failure
- o Loss of support due to insufficient seat width
- o Pier column failures

Each retrofit system selected must be evaluated to ensure that it does not transfer excessive force to other less-easily inspected and repaired components. All retrofit components should be designed to the standards listed here but, whenever possible, not less than the standards for the design

of new structures. Reduced standards may be used when the use of full design standards is not practical or economically feasible, and partial strengthening significantly reduces the risk of unacceptable damage. Further guidance and illustrations of the retrofit systems are found in FHWA, May 1995. The following are examples of systems that can be used:

- o *Replacement of bearings:* Certain types of bearings, such as tall steel rocker bearings, have performed poorly during past earthquakes because of their low resistance to horizontal loads. Replace these bearings with modern bearing types such as steel laminated elastomeric bearings or multi-rotational bearings such as pot or disc bearings.
- o *Bearing restrainers:* Transverse and longitudinal restrainers will keep the superstructure from sliding off the bearings. Conditions that are particularly vulnerable include tall concrete pedestals that serve as bearing seats for individual girders, and bearing seats where the transverse distance between the bearing and the edge of the seat is small.
- o *Bearing seat extension:* Extension of bearing seats may be a feasible retrofit measure in certain situations. Since high forces may be imposed on these extensions, it is recommended that they be supported directly on a foundation structure when possible. All bearing seat extensions should provide a final minimum seat width equal to or greater than the specified value given in Section 3.7.2 Seismic Analysis.
- o *Pier column retrofitting:* Under seismic loads, high shear stresses develop between column and cap beam or between column and footing. Therefore, increased transverse confinement should be located within the column end regions. Refer to AASHTO Standard Specification Section 6.

Be aware that retrofit schemes for increasing confinement may redistribute moments and shears resulting in overstress in other members of the pier, i.e., footing and bent caps.

Five retrofitting systems are commonly used to retrofit concrete columns. The following systems laterally confine the concrete and increase the member's strength and ductility.

1. Preformed jacketing: This technique uses steel or FRP plates or shells to passively confine the column.
2. Prestressed wire wrapping: This technique uses wire wrapped around the column under tension to actively confine the column.

3. Composite fiberglass/epoxy wrapping: This technique involves an FRP fabricated on-site and wrapped around the column. When the FRP cures, the system confines the column. ||
4. Concrete jacketing: This involves the addition of a thick layer of reinforced concrete. ||
5. External hoops: This technique uses external hoops that are tensioned around columns using turnbuckles. ||

10.10 Buried Structures

MaineDOT has hundreds of steel culverts that are considered minor spans or bridges. Many of these steel culverts are reaching the end of their design life of 45 to 55 years. Instead of culvert replacement, another option to consider is culvert rehabilitation. MaineDOT began rehabilitating culverts in the early 1990s. |

If culvert rehabilitation is a feasible option, the final decision to rehabilitate or replace usually depends upon one of the following issues:

- o Maintenance of traffic
- o Right-of-way impacts
- o Utility impacts
- o Environmental impacts, including fish passage (short & long term)
- o Constructability
- o Maintenance
- o Cost (first cost and life cycle)

10.10.1 *Invert Lining*

Culvert invert lining consists of placing a minimum of 5 inches of reinforced concrete in the bottom and sides of a pipe or pipe arch that has a rusted or missing bottom. The Contractor has the option of using shotcrete or cast in place concrete. The top of the concrete invert lining should extend a minimum of 6 inches above the limit of the rust line or the proposed location of shear studs, whichever is higher. The estimated life for a concrete invert lining is about 25 years.

Culvert invert lining is a feasible alternative if all of the following statements are true:

- o The culvert has not distorted significantly.
- o The top plates and side plates for the culvert are in good condition. Some very minor rusting in spots is acceptable as long as the areas are painted with zinc-rich paint.
- o The alignment and/or road width will not change in the next 20 years +/-.
- o The hydraulic capacity is adequate even with the reduction in opening area. The Designer should check the reduced opening for its flowing full capacity and its ability to handle Q50. A reduced design flow may be acceptable depending on the individual project, and good engineering judgment is required to evaluate the adequacy of the reduced opening.
- o Fish passage can be maintained when necessary. This may involve the use of grade control structures, weirs, baffles, or other methods. Refer to Section 2.3.8.6 Fish Passage.
- o The culvert has adequate cover.
- o The rust line does not extend more than half way up the side of a pipe or much above the corner plates for a pipe arch.

A site visit for a possible culvert rehabilitation project should include measurements of the rust line height and the lowest elevation at which shear studs can be welded.

10.10.2 Sliplining

Sliplining consists of installing a slightly smaller diameter pipe or pipe arch inside an existing culvert. The gap (i.e. annular space) between the new and existing culvert is filled with grout or flowable fill. Typically an aluminum pipe or pipe arch will be used inside an existing rusted steel culvert. The estimated life for a sliplining is about 75 years. As a general rule, sliplining is a feasible alternative if the all of the following statements are true:

- o The culvert has not distorted significantly.
- o The alignment and/or road width will not change in the next 20 years +/-.
- o The hydraulic capacity is adequate even with the reduction in opening area. The Designer should check the reduced opening for its flowing full capacity and its ability to handle Q50. A reduced design flow may be acceptable depending on the individual project, and good

engineering judgment is required to evaluate the adequacy of the reduced opening.

- o Fish passage can be maintained when necessary. This may involve the use of grade control structures, weirs, baffles, or other methods. Refer to Section 2.3.8.6 Fish Passage.
- o The culvert has adequate cover.

Sliplining should be given serious consideration in the following situations:

- o High traffic volumes
- o Lack of a detour route or a reasonably short detour
- o Deep fills (8 feet or more over the culvert)

If there is any doubt that distortion of the culvert may preclude the use of sliplining, the interior of the culvert should be surveyed as discussed in Section 2.5.2 Field Survey.

References

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National Highway Institute, 1993, Seismic Design of Highway Bridges, *NHI Course No. 13048*, Participant Workbook, prepared by Imbsen Associates, June

APPENDICES



Casco Bay Bridge, Portland-South Portland



Artist Bridge, Newry

APPENDICES

Appendix A	Bridge Nomenclature	A-1
A.1	Terminology.....	A-1
A.1.1	Acronyms	A-1
A.1.2	Approaches	A-4
A.1.3	Contract.....	A-8
A.1.4	Environmental	A-12
A.1.5	General.....	A-14
A.1.6	Geotechnical	A-18
A.1.7	Structures	A-20
A.1.8	Welds	A-33
A.2	Drawings.....	A-36
Appendix B	PDR Forms (Section Removed).....	B-1
Appendix C	Hydrology/Hydraulics	C-1
C.1	Transposed Discharge Methods.....	C-1
C.1.1	Transposed Discharge Method #1.....	C-1
C.1.2	Transposed Discharge Method #2.....	C-2
C.1.3	Transposed Discharge Method #3.....	C-2
C.2	Discharge Adjustment Factors.....	C-3
C.3	Forms	C-3
Appendix D	Standard Notes	D-1
D.1	Title Sheet	D-1
D.2	General Construction Notes	D-3
D.3	Standard Notes Abutments.....	D-6
D.4	Standard Notes Piles	D-7
D.5	Standard Notes Piers	D-8
D.6	Standard Notes Seal Cofferdams	D-9
D.7	Standard Notes Structural Steel	D-10
D.8	Standard Notes Precast Concrete Superstructures	D-12
D.9	Standard Notes Superstructures	D-13
D.10	Standard Notes Elastomeric Bearings	D-15
D.11	Standard Notes HLMR Bearings	D-16
D.12	Standard Notes Structural Plate Structures	D-17
D.13	Standard Notes Drilled & Anchored Bolts and Reinforcing Steel ..	D-18
D.14	Standard Notes Precast Concrete Arches or Boxes	D-19
D.15	Standard Notes Prefabricated Concrete Modular Gravity Wall	D-20
D.16	Standard Notes CIP Box Culverts	D-21
Table C-1	Discharge Adjustment Factors	C-3

APPENDICES

Figure A-1 Abutment, Pier, Superstructure A-36
Figure A-2 Superstructure and Substructure A-37
Figure A-3 Gravity Abutment A-38
Figure A-4 Full Height Cantilever Abutment..... A-39
Figure A-5 Stub Abutment A-40
Figure A-6 Open Abutment A-41
Figure A-7 Solid Pier..... A-42
Figure A-8 Types of Piers A-43

Appendix A Bridge Nomenclature

A.1 Terminology

A.1.1 Acronyms

Acronyms	
AADT	average annual daily traffic
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ACM	Associated Constructors of Maine
ADA	Americans with Disabilities Act
ADT	Average daily traffic
AGC	Associated General Contractors of America
AGC	Association of General Contractors
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
ANSI	American National Standards Institute
AREA	American Railway Engineering Association
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
ATSSA	American Traffic Safety Services Association
AWPA	American Wood-Preservers' Association
AWS	American Welding Society
BLCCA	bridge life cycle cost analysis
BMP	Best Management Practices for Erosion and Sediment Control
BPR	Bureau of Public Roads
BTIP	Biennial Transportation Improvement Program
CIP	cast-in-place
CL	centerline
CRF	critical rate factor
CRSI	Concrete Reinforcing Steel Institute
DHV	design hour volume
DTM	digital terrain model
EA	Environmental Assessment
EIS	Environmental Impact Statement
FAA	Federal Aviation Administration
FEM	finite element method

Acronyms	
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FONSI	Finding Of No Significant Impact
FOS	Factor of Safety
fps	feet per second
FRP	fiber-reinforced polymer
GPR	ground-penetrating radar
GPS	Global Positioning System
GRS wall	geosynthetic-reinforced soil wall
HLMR bearing	high load multi-rotational bearing
HPC	high performance concrete
HUD	U.S. Department of Housing and Urban Development
ITE	Institute of Transportation Engineers
LCCA	life cycle cost analysis
LFD	Load Factor Design
LRFD	Load Resistance Factor Design
LURC	Land Use Regulation Commission - Maine
MaineDOT	Maine Department of Transportation - current
MDEP	Maine Department of Environmental Protection
MDOT	Maine Department of Transportation - outdated
MELT	Modified Eccentric Loader Terminal
MEMA	Maine Emergency Management Agency
MHHW	mean higher high water
MHW	mean high water
MHPC	Maine Historic Preservation Commission
MLLW	mean lower low water
MLW	mean low water
mph	miles per hour
MSE wall	mechanically stabilized earth wall
MSL	Multiple Service Level
MTL	mean tide level
MUTCD	Manual on Uniform Traffic Control Devices
NAVD	North American Vertical Datum
NC	normal crown
NCHRP	National Cooperative Highway Research Program
NEBT	New England Bulb Tee
NEPCOAT	Northeast Protective Coating Committee
NGVD	National Geodetic Vertical Datum
NHS	National Highway System
NOS	National Ocean Service
NRCS	Natural Resources Conservation Service
NSBA	National Steel Bridge Alliance
NTDE	National Tidal Datum Epoch

Acronyms	
PCE	pre-construction engineering
PCE-C	pre-construction engineering - conceptual
PCE-F	pre-construction engineering – feasibility study
pcf	pounds per cubic foot
PCI	Precast Concrete Institute
PCMG wall	prefabricated concrete modular gravity wall
PDR	Preliminary Design Report
PIN	Project Identification Number
PL	Performance Level
PQR	Procedure Qualification Record
PS&E	plans, specifications, and estimate
psf	pounds per square foot
psi	pounds per square inch
PTFE	polytetrafluoroethylene - also known as Teflon
PVC	polyvinyl chloride
QA	Quality Assurance
QC	Quality Control
QCP	Quality Control Plan
QPL	Qualified Products List
RC	remove crown
RFI	Request for Information
ROW	right-of-way
R/W	right-of-way (alternate designation)
SCF	Site Coefficient Factor
SCS	Soil Conservation Service
SEWPCP	Soil Erosion and Water Pollution Control Plan
SI&A	Structure Inventory and Appraisal
SPC	Seismic Performance Category
SPCCP	Spill Prevention Control and Countermeasure Plan
STIP	Surface Transportation Improvement Program
TCP	Traffic Control Plan
TL	Test Level
TRB	Transportation Research Board
tsf	tons per square foot
U.S.G.S.	United States Geologic Survey
VECP	Value Engineering Change Proposal
WL	working line
WP	working point
WPS	Welding Procedure Specification

A.1.2 Approaches

Approaches	
approach	A general term referring to the roadway at either end of a bridge.
arterial	Routes that focus on high mobility of traffic with minimal or limited land access, often at relatively high speeds.
backslope	In an approach roadway, the upward sloping earth cut beyond the ditch line.
base course	One or more layers of specified material thickness placed on a subbase or a subgrade to support a surface course.
berm	The outside edge of the approach roadway shoulder and top of the sideslope.
cross-slope	The slope of the pavement surface perpendicular to the centerline.
ditch	A channel formed at the bottom of the sideslopes for open drainage.
embankment	The raised structure of earth supporting the approach roadway.
foreslope	An alternate designation for sideslope.
grade	The slope of a surface to the horizontal, often along a centerline.
guardrail	A vertical element near mid-height of the vehicles used to guide traffic.
gutterline	The intersection of the cross-slope and curb where drainage concentrates.
highway	A general term denoting a public way for purposes of vehicular travel, including the entire area within the right-of-way.
highway curb	A low barrier constructed along the edge of a roadway, serving to guide the wheels of an errant vehicle and to control drainage.
lane	A strip of roadway intended to accommodate a single line of vehicles.
major collector	A route that serves a county seat that is not on an arterial route, a larger town not directly served by the higher systems, or other traffic generator of equivalent intracounty importance, such as a consolidated school, a shipping point, a county park, or an important mining or agricultural area.

Approaches	
minor collector	A route that is spaced at an interval consistent with population density to accumulate traffic from local roads and bring all developed areas to within a reasonable distance of collector roads, provides service to remaining small communities, or links a locally important traffic generator with its rural hinterland.
National Highway System	A system of interstate highways and major collectors specifically designated by FHWA. It includes the interstate system, other urban and rural principal arterials, highways that provide motor vehicle access between the NHS and major intermodal transportation facilities, the defense strategic highway network, and strategic highway network connectors.
normal crown	The typical cross section on a tangent section (i.e., no superelevation).
pavement structure	The combination of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed.
profile	An elevation view of the longitudinal centerline of construction.
profile grade	The trace of a vertical plane intersecting the top of the wearing surface, usually along the longitudinal centerline of the roadbed. Profile grade means either elevation or gradient of such trace according to the context.
remove crown	A superelevated cross section which is sloped across the entire traveled way in the same direction and at a rate equal to the typical cross slope on a tangent section (e.g., 2.0%).
riprap downspout	An open channel armored with riprap used to convey stormwater runoff from the top of a steep slope to the bottom of the slope in a controlled and stable manner.
road	A general term denoting a public way for purposes of vehicular travel, including the entire area within the right-of-way.
roadbed	The graded portion of a highway within top and side slopes, prepared as a foundation for the pavement structure and shoulders.
roadside	A general term denoting the area adjoining the outer edge of the roadway. Extensive areas between the roadways of a divided highway may also be considered roadside.

Approaches	
roadside development	Those items necessary to complete the highway that provide for the preservation of landscape materials and features; the rehabilitation and protection against erosion of all areas disturbed by construction through seeding, sodding, mulching, and the placing of other ground covers; and such suitable planting and other improvements as may increase the effectiveness and enhance the appearance of the highway.
roadway	The combined traveled way and shoulders of a public way intended for passage of vehicular traffic. A divided highway has two or more roadways.
rural	A term used to denote all areas that are not designated as urban.
shoulder	The portion of the road or roadway that is contiguous with the traveled way and that is provided for accommodation of stopped vehicles, emergency use, and lateral support of base and surface courses. Where guardrail is installed on the approach roadway, the shoulder area between the traveled way and the face of guardrail is the "design" or "usable" shoulder. The shoulder area between the face of guardrail and the berm is the "berm offset."
sideslope	In an approach roadway, the downward sloping embankment beyond the outside edge of the shoulder. The top of the sideslope is the berm. The bottom of the sideslope is the "toe of slope".
station	A distance of 100 feet used in the layout of centerlines, working lines, and other reference points on a project.
street	A general term denoting an urban public way for purposes of vehicular travel, including the entire area within the right-of-way.
subbase	Layers of specified material thickness placed on a subgrade to support a base course.
subgrade	The top surface of a roadbed upon which the pavement structure, shoulders, and curbs are constructed.
superelevation	Superelevation is the amount of cross slope or "bank" provided on a horizontal curve to help counterbalance the outward pull of a vehicle traversing the curve. The maximum rate of superelevation depends on several factors including climatic conditions, terrain conditions, and type of area (rural or urban).

Approaches	
superelevation axis of rotation	The superelevation axis of rotation is the line about which the pavement is revolved to superelevate the roadway. This line will maintain the normal highway profile throughout the curve, and it is known as the construction centerline or control edge.
superelevation rollover	Superelevation rollover is the algebraic difference between the travel lane cross slope and shoulder cross slope on the outside of a horizontal curve.
superelevation runoff	AASHTO defines superelevation runoff as the change in cross slope from the end of tangent runout (adverse crown removed) to a section that is fully superelevated.
superelevation transition length	The superelevation transition length is the distance required to transition the roadway from a normal crown section to the full superelevation needed. Superelevation transition length is the sum of the tangent runout and superelevation runoff.
surface course	The top layer(s) of a pavement structure designed to accommodate the traffic load, resist skidding, traffic abrasion, and the disintegrating effects of climate. This layer is sometimes called the "wearing course".
tangent runout	AASHTO defines tangent runout as the change from a normal crown section to a point where the adverse cross slope of the outside lane or lanes is removed (i.e., the outside lane(s) is level).
townway	A road that is locally owned and maintained.
urban	A term used to denote a place within boundaries set by the responsible State and local officials having a population of 5000 or more.

A.1.3 Contract

Contract	
addendum	See bid amendment.
award	The execution of the contract by MaineDOT, conditioned upon the successful bidder's performance of all pre-execution requirements of the bid documents.
best value procurement process	A process for procuring contractual services in which price is only one of several factors used in determining the successful proposer.
bid	The offer by a bidder on forms prescribed by MaineDOT to perform the work in conformity with all provisions of the bid documents for the price(s) set forth.
bid amendment	A change to the bid documents issued by MaineDOT after advertisement and before the bid opening.
bid documents	Documents issued by MaineDOT to solicit bids from Contractors. Bid documents generally include the Notice to Contractors, plans, Standard Specifications, Special Provisions, bidding instructions, and any bid amendments issued by MaineDOT. Documents attached to or referenced in the bid documents are part of the bid documents.
bid opening	The date and precise time by which the bidder must deliver its bid to be publicly opened and read as specified in the Notice to Contractors or any applicable bid amendment.
change order	See contract modification.
conform or conformity	The performance of an item of work in strict compliance with all applicable provisions of the contract.
contract	All documents affecting the respective rights and responsibilities of MaineDOT and the Contractor. These documents include, but are limited to, the contract agreement, the Notice to Contractors, plans, MaineDOT's Standard Specifications and Standard Details, Special Provisions, bid amendments, contract modifications, geotechnical information, permits, bid escrow documentation, the Contractor's bid prices, and all documents incorporated by reference.
contract modification	A general term describing a formal change to a contract. Types of contract modifications include; change orders, extra work orders, resident work orders, and supplemental agreements.

Contract	
critical path	The sequence of activities from the project start to its completion having the greatest cumulative elapsed time, thereby determining the minimum time duration of the entire project. The critical path is identified by the sequence of those activities with the least float.
Department	An alternate designation for MaineDOT.
design-build contract	A contract in which the Contractor is responsible for both design and construction requirements under the contract. In a design-build contract, the Contractor is procured through a best-value procurement process using a Request for Proposals and evaluation of submitted proposals using price as one of several evaluation factors.
extra work	Work that is outside the scope of the contract and that MaineDOT determines is necessary.
extra work order	See contract modification.
force account work	Prescribed work paid on the basis of actual costs and additives.
incentive or disincentive payment	An adjustment to the contract price of a predetermined amount for each day the work is completed ahead of or behind the contract time, contract completion date, or some specified intermediary milestone. A disincentive is not a penalty, but an estimate of user and other costs incurred by the people of the State of Maine.
incidentals	The terms "incidentals" and "incidental to the contract" mean items that are accessory to or incorporated into the work and that have no separate pay item. Unless otherwise provided in the contract, the cost of incidentals shall be included in the Contractor's prices for the pay items. There will be no separate payment.
liquidated damages	An amount due and payable to MaineDOT by the Contractor, normally realized through a reduction of amounts to be paid to the Contractor. Said amount is calculated by multiplying a daily amount set forth in the contract by the number of days the work remains incomplete after the contract completion time has expired.
major item	An individual pay item that constitutes 10% or more of the amount of the awarded contract, calculated using the Contractor's bid prices and the estimated quantities contained in the bid documents.
minor item	All pay items that are not major items.
order	A directive from MaineDOT requiring compliance by the Contractor.

Contract	
partnering	Partnering is a process of voluntary structured communication between MaineDOT, the Contractor, its principal Subcontractors and suppliers, and other project stakeholders for the purpose of improving efficiency and minimizing disputes.
pay item	An item of work set forth in the schedule of Items for which the Contractor must provide a price.
proposal	The response to a Request for Proposal. Proposals will normally be requested for anticipated best value procurements. See design-build contract, Request for Proposal and best value procurement. In another context, sometimes MaineDOT's solicitation for bids is called a bid proposal.
punch list	A written list of all items that must be completed or corrected before the physical work is complete. This list is prepared by MaineDOT and given to the Contractor.
quality assurance	All planned and systematic operations to ensure that the operation, material, and/or end product meets specifications. Quality assurance includes approval and oversight of the Contractor's Quality Control Plan; review of inspector, sampler, tester, and laboratory qualifications; inspection for conformity with contract requirements; Contractor quality control; acceptance testing; and independent assurance.
quality control	Planned and specified actions or operations necessary to produce an end product that conforms to the quality requirements of the contract. Unless otherwise specified, Quality control includes inspection and testing for process control to the extent determined necessary by the Contractor. Quality control is also referred to as process control.
Quality Control Plan	The program and documentation of that program, approved by MaineDOT, which specifies the actions, inspection, sampling, and testing necessary to keep production and placement operations within specifications, including provisions to quickly determine when an operations becomes out of control and those actions that the Contractor will take to restore compliance.
Request For Information	A written request for information or clarification submitted by the Contractor to the Construction Resident.
Request For Proposal	MaineDOT's solicitation for proposals in a best value procurement process, such as when soliciting for an anticipated design-build contract.

Contract	
resident work order	See contract modification.
schedule of items	A list of items of work provided in the bid documents for which the Contractor must provide prices.
Special Provision	Revisions to the Standard and/or Supplemental Specifications applicable to an individual project or contract.
Specifications	A written or electronic textual compilation of provisions and requirements for the performance of the work, including incorporations by reference.
Standard Details	Detailed drawings published and approved by MaineDOT for general application and repetitive use.
Standard Specifications	The Standard Specifications for Construction published and approved by MaineDOT for general application and repetitive use on projects.
supplemental liquidated damages	Liquidated damages for additional costs resulting from Contractor's failure to complete a specific work item, phase, or milestone within the time specified in the contract for that item. Supplemental liquidated damages are in addition to and separate and distinct from liquidated damages.
Supplemental Specifications	Approved additions or modifications to the Standard Specifications.
Supplemental Standard Details	Approved additions or modifications to the Standard Details.
Value Engineering Change Proposal	A proposal made by a Contractor after contract execution that is intended to produce cost savings without impairing essential characteristics of the project including function, serviceability, safety, durability, maintainability, and aesthetics, all as determined by MaineDOT.

A.1.4 Environmental

Environmental	
check dam	Small dams usually consisting of dumped stone constructed across a swale or drainage ditch to reduce the flow velocity of concentrated stormwater runoff.
dredge material or dredge spoils	“Dredge material” means sand, silt, mud, gravel, rock or other sediment or material that is removed from beneath any surface water. The term, “beneath any surface water” has been interpreted by the MDEP to mean that area that falls beneath the plane bounded by the normal high water line of any stream, river, brook, pond, lake, vernal pool, etc. Note that the entire area of dredge material removal could be dry at the time of excavation.
erosion	The detachment and movement of soil particles by the action of water, ice, gravity, and/or wind.
erosion control blanket	A sheet that is composed of biodegradable material such as jute matting, excelsior wood fiber, coconut fiber, straw or interwoven paper strips, and a netting made of a biodegradable polypropylene or extruded plastic.
floating boom-supported silt fence	A temporary silt fence barrier that is installed in the water and supported by a flotation boom along with weighting devices to hold the fence in an approximately vertical submerged position from the surface of the water to the design depth.
in-stream work	Any activities conducted in the water.
mulch	A material usually composed of hay, straw, wood waste compost/bark, crushed stone, or cellulose fiber that is placed on disturbed areas to prevent erosion.
riprap	A blanket of sizable angular stones placed along a slope or in a watercourse to hinder erosion. Types of riprap include plain, heavy, and hand laid.
runoff	Rainfall not absorbed by the soil.
sediment	The finely divided solid material that settles to the bottom of water or is suspended in the water.
sedimentation	The deposition of soil particles that are dislodged during the erosion process and transported by water or wind. It occurs when the velocity of the wind or water is insufficient to maintain suspension of the soil particles.
silt fence	A type of temporary sediment barrier constructed of geotextile fabric and posts that is installed parallel to the toe of a slope being disturbed.

Environmental	
slope stabilization	The use of vegetation and/or structural materials to stabilize and protect slopes of roadways, streams, brooks, rivers, lakes, tidal areas, or excavated channels against scour and erosion from flowing water.
stream diversion	A conduit or small diversion ditch used to divert the base flow of a perennial or intermittent stream around a construction area.
swale	A low tract of marshy land.
wetlands	Areas inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.

A.1.5 General

General	
aggregate	Inert material such as sand, gravel, broken stone, crushed stone, or a combination of any of these materials.
alignment	The baseline for construction of a bridge and its approach roadway, described horizontally by a series of tangents and circular arcs, and vertically by a series of tangents and parabolic curves.
bearing centerline	The construction reference line that passes through the center of a line of bearings on an abutment or pier.
common borrow	Standard fill material used on approaches where a more free-draining material such as granular borrow is not required.
common excavation	Work consisting of the removal and disposal of all earth material encountered in excavating for permanent stream channel diversion, channel widening, when designated on the plans, outside the limits of structural excavation, or other classifications.
compact area	A urban area where the population exceeds 7500 inhabitants; or where the population is less than 7500, but more than 2499 inhabitants in which the ratio of people whose place of employment is in a given municipality to employed people residing in that same municipality is 1.0 or greater.
conduit	A pipe used for receiving and protecting wires or cable.
construction centerline	Typically, the alignment of the roadway from which all construction layouts are described. (See alignment.)
construction easement	A right acquired by MaineDOT to use or control property, outside of the established right-of-way.
construction limit line	A line, usually outside of the right-of-way, within which the Contractor may work and outside of which work may not be performed.
drainage	The system of pipes, drainage ways, ditches, and structures by which surface or subsurface waters are collected and conducted from the highway area.
drawings	See plans.
elevation	A scale drawing of the side, front, or rear view of a structure.
elevation	The vertical distance above a horizontal reference plane.
geometrics	The physical location (horizontally and vertically) and shape of the object under consideration.

General	
granular borrow	Fill material consisting of granular material, typically used next to or under structures.
gravel borrow	Fill material consisting of gravel.
hydraulics	The science concerned with the behavior and flow of liquids, especially in pipes and channels. Used to design the size of bridge openings.
hydrology	The science concerned with the occurrence, distribution, and circulation of water on the earth, including precipitation, runoff, and groundwater. Used to determine the design flow rates for sizing bridge openings.
Inspector	An authorized representative of the Resident assigned to make detailed inspections of the work to determine compliance with the contract.
longitudinal	The direction along the length of a bridge, following the centerline of construction.
material	Any substance specified for use in the construction of the project and related approaches.
normal	Any line placed perpendicular to a tangent or radial to a curve.
permits	Permits granted to MaineDOT for the project. Permits often required include a U.S. Coast Guard permit and environmental permits such as Natural Resources Protection Act (NRPA) permit from MDEP and Army Corps of Engineers permit.
plans	When the context so indicates, “plans” mean applicable construction drawings including plan, profile, typical cross sections, working drawings, Standard Details, Supplemental Standard Details, and supplemental drawings or exact reproductions thereof or electronically displayed equivalents, that show the location, character, dimensions, and details of the work. Where the context so indicates, “plan” may also mean a detailed process, program, or method worked out beforehand for the accomplishment of an objective. Examples include QCP, the SEWPCP, the TCP, Safety Plan, and project-specific emergency planning.
Program	The specific working unit within MaineDOT’s Bureau of Project Development within which a particular MaineDOT project is developed, designed, and administered. Such Programs include the Regional Program, the Urban and Arterial Highway Program, the Bridge Program, and the Multimodal Program.

General	
project	The bridge, highway, railroad, pier, airport, building, bike path, pedestrian path, or other infrastructure improvement being constructed, rehabilitated, or repaired, together with all appurtenances and Incidentals.
project limits	Areas within the right-of-way or construction limit lines shown on the plans or otherwise indicated in the contract. If no project limits are indicated in the contract, the project limits shall be the area actually occupied by the bridge, highway, or other infrastructure before construction extending to and including (A) the area outside the shoulders and ditch lines and within any landmarks or historic features such as fences, fence posts, tree rows, stone walls, corner stones, or other monuments indicating the boundary line, or (B) in the absence of any landmarks or historic features, sidewalks, shoulders, and ditch lines to the top of cuts or toe of fills.
Project Manager	MaineDOT's duly authorized representative for overall coordination of the project.
ProjEx	MaineDOT's Program and Project Management System developed for transportation program development, "cradle to grave" project planning and delivery, and enterprise wide access to project information.
Resident	MaineDOT's on-site representative.
right-of-way	A general term denoting land, property, or interest therein, usually in the form of a strip, acquired for or devoted to the Project or other purposes.
utility facilities	All structures, facilities, equipment, and all appurtenances thereto used by utility companies including, but not limited to, poles, wires, support poles, guys, anchors, water pipelines, sewer pipelines, gas pipelines, all other pipelines, fire alarms, service connections, meter boxes, valve boxes, light standards, cableways, conduits, signals, and manholes.
working drawings	Plans, sketches, or drawings provided by the Contractor or its Subcontractors, vendors, or fabricators for the purpose of supplementing the plans provided in the bid documents and being necessary to demonstrate that the work will comply with the contract and meet the intent of the contract. Working drawings shall be of sufficient detail to meet the purpose set forth in the preceding sentence. Examples include shop drawings, erection plans, falsework plans, cofferdam plans, and bending diagrams for reinforcing steel.

General	
working line	A horizontal tangent established on a curved alignment to facilitate the layout and construction of a bridge.
working point	The theoretical intersection of two lines, used as a layout reference point.

A.1.6 Geotechnical

Geotechnical	
bedrock	The solid mineral rock mass underlying and/or outcropping from soil or any other loose agglomerate surficial cover.
clay	A fine-grained, firm soil where particles are less than 0.002 mm in size and exhibit cohesive and plastic behavior when wet.
downdrag	A force that may be induced on a pile or buried structure where the soil deposit in which the pile or structure is installed is subject to settlement.
foundation	The part of a substructure which bears directly on either soil or bedrock. A foundation may be shallow, as with a spread footing, or deep, as with steel piles, drilled shafts, etc.
geophysics	Any nondestructive method used to determine the subsurface conditions/characteristics or unexposed limits of a buried structure/substructure. Examples include seismic refraction and ground penetrating radar.
geosynthetic	A flexible, synthetic, polymeric material used for separation, reinforcement, filtration, drainage, or containment. Examples include geotextiles, geogrids, and geomembranes.
geotechnical information	Boring logs, soil reports, geotechnical design reports, foundation design reports, hazardous waste assessments, and other records or reports of subsurface conditions.
geotextile	A polymeric material formed by the weaving or knitting (woven) or matting (nonwoven) of synthetic fibers into a flexible, continuous sheet. Geotextiles may be constructed from polyester (PET), polypropylene (PP), or high density polyethylene (HPDE).
gravel	Soil where the particles are less than 75 mm and greater than 4.75 mm in size and are cohesionless, nonplastic, and granular in nature.
ledge	A narrow, shelflike ridge or rock protrusion.
lightweight fill	Any construction material having a unit weight less than that of traditional soil backfills, such as lightweight foam concrete, geofoam, and tire shreds.
liquefaction	A soil condition caused by an intense load, of very short duration, resulting in a sudden increase in pore-water pressure and loss of shear strength. Liquefaction can result from cyclic loading associated with a seismic event.

Geotechnical	
prefabricated vertical drain	A geotextile consisting of a grooved plastic or paper core covered by a membrane placed vertically in the ground to produce a “wick” for faster movement of water in the soil structure. Also called wick drains, they accelerate settlement and the gain in strength of soft cohesive soils.
preload	Placement of materials to improve poor foundation soils in advance of construction of a bridge or other facility. Materials may consist of soil, rock, or other heavy materials.
rock excavation	Work consisting of the removal of hard igneous, metamorphic, and sedimentary rock that cannot be excavated without drilling and blasting or drilling and splitting. It also consists of excavating all boulders, solid mortared stone masonry, or concrete masonry, each with a volume of 2 CY or more.
sand	Soil where particles are less than 4.75 mm and greater than 0.075 mm in size and are cohesionless, nonplastic, and granular in nature. Sand is finer than gravel, but coarser than silt.
silt	Soil where particles are less than 0.075 mm and greater than 0.002 mm in size and are cohesionless and granular in nature with low plasticity. Silt is finer than sand, but coarser than clay.
soil	The relatively loose agglomeration of mineral, organic materials, and sediments found above bedrock.
soil envelope	The zone of controlled soil backfill around a culvert structure required to ensure anticipated performance based on soil-structure interaction considerations.
structural excavation	Work consisting of the removal, hauling and backfilling, and disposal of all material encountered for the installation and construction of drainage and minor structures and for major structures.
surcharge	A load used to model the weight of earth fill or other loads applied to the top of the retained material. Typically used behind new abutment or wall locations and usually removed prior to construction.
surficial	Of, relating to, or taking place on the earth’s surface.

A.1.7 Structures

Structures	
abutment	The supporting structure at either end of a traditional bridge, which also serves to retain the approach roadway embankment. Types include cantilevered, deep or full height, gravity or mass, integral, semi-integral, or stub.
ancillary product	A product not subject to calculated tensile stress from live load and not welded to main members in tension areas. Types include drainage components, expansion devices, curb plates, bearings, hand rails, cofferdams, and sheet piling.
approach slab	A concrete slab placed in the approach roadway and resting on the abutment backwall, distributing traffic loads and preventing settlement of the roadway behind the abutment.
appurtenance	A component such as curb, parapet, railing, barrier, divider, or sign and lighting post that is attached to the deck.
armored joint	Alternate name for expansion device.
backwall	The top portion of an abutment above the breastwall, serving primarily as a retaining wall but also supporting the approach slab and often the end of the bridge deck. When the top of backwall extends to finished grade, it supports one side of the expansion device at the end of the superstructure.
barrier parapet	The solid concrete vertical wall of a concrete barrier rail. Often includes a metal rail mounted on top.
batter	A slope, as of the outer side of a wall, that recedes from bottom to top.
batter pile	A pile driven at an angle inclined to the vertical to provide higher resistance to lateral loads.
beam	In a traditional bridge, a main load-carrying member spanning longitudinally between supports. (See girder.)
bearing	A device installed on the bridge seat to support a beam or girder, allowing for deflection and expansion of the superstructure, and the transmission of loads to the substructure. Types include elastomeric, HLMR, pedestal or rocker, and spherical.
bearing plate	Used to transmit loads from the superstructure to the substructure.
bituminous concrete	Asphalt concrete used on roadway and bridge riding surfaces.

Structures	
blocking	The rectangular section of concrete placed between the top of a beam and the bottom of the deck slab. Blocking dimensions are used to set bottom-of-slab elevations.
box beam	A precast, prestressed concrete hollow box-shaped beam. Box beams are typically 3 feet or 4 feet wide. They may be installed as butted beams post-tensioned together, or as spread beams with a CIP composite concrete slab.
box culvert	A buried structure, typically of aluminum plates or concrete, with a generally rectangular shaped opening.
box girder	A box-shaped girder of concrete or steel, usually multi-celled and with several interior webs.
brace	A structural member that provides stiffness to a frame.
breastwall	The lower wall portion of an abutment, providing the support for the superstructure.
bridge	A structure having a clear span of 20 feet or more measured horizontally at the elevation of the bridge seats along the centerline of the road or in case of multiple spans when the combined clear spans equal or exceed 20 feet.
bridge culvert	A buried structure with a span of 20 feet or more, generally of steel or aluminum, consisting of plates bolted into a round, elliptical or oblate shaped opening.
bridge curb	A low barrier constructed along the edge of a bridge deck, serving to guide the wheels of an errant vehicle and to control drainage.
bridge drain	A fabricated unit installed in a bridge deck to remove surface water.
bridge length	The length of a bridge structure is the overall length measured along the construction centerline back to back of backwalls of abutments, if present; otherwise end to end of the bridge floor; but in no case less than the total clear opening of the structure.
bridge rail	A railing system installed on the edges of a bridge deck or along a retaining wall to provide protection to the vehicles and pedestrians crossing the bridge.
bridge railing	The horizontal member elements.
bridge roadway width	The clear width measured at right angles to the longitudinal centerline of the bridge between the bottoms of curbs or between face of rails, whichever is less.
bridge seat	The top horizontal surface of an abutment breastwall or pier upon which the superstructure is supported.
brittle fracture	A sudden failure of an element prior to plastic deformation typically occurring at a sharp change of section properties. Also called low energy fracture.

Structures	
buckling	Deflection in an axially loaded member under compression.
bulb tee girder	A precast, prestressed concrete girder whose cross section somewhat resembles the letter “T” with a bulb shape on the bottom. MaineDOT uses NEBT standard shapes.
buried structure	Any culvert-type bridge structure deriving its strength from the condition of being surrounded by soil. (See traditional bridge and rigid frame.)
butt joint	The joint between two pieces of metal in the same plane that have been bolted or welded together.
butterfly wingwall	A wing that extends straight out from the abutment and has no footing of its own.
camber	The curving of a beam to compensate for deflections that occur when a load, such as a bridge deck, is placed on it.
cantilevered abutment	An abutment where the wall is designed as a cantilevered beam. See Figure A-4.
cap	The topmost portion of a pier, when supported by a shaft, columns or exposed piles. The top of the cap is the pier bridge seat.
catch basin	A precast concrete structure that is part of a closed drainage system with an inlet grate and culvert outlet.
cathodic protection	A system used to protect steel from the deteriorating effects of roadway salts or salt water. A passive system typically involves a sacrificial core such as zinc, and requires no external source of energy. An active system typically has an external energy source.
channel shape	An AISC rolled section in the shape of a “C”, “M”, or channel.
Charpy V-Notch Test	An impact test used to determine the notch toughness of a material. The test is performed at specified temperatures, to provide information about the toughness of the metal and the temperature at which it can be expected to fail in a brittle manner.
chord	Primary members located at either the top or bottom plane of a truss.
CIP concrete	Concrete placed in its final location in the structure while still in a plastic state.
class A concrete	Concrete typically used for standard CIP applications.
class LP concrete	Low permeability concrete usually containing silica fume, typically used in locations that are exposed to road salts or salt water.
class P concrete	Concrete typically used for precast concrete applications.
class S concrete	Concrete used in seals.

Structures	
clear span	The face-to-face distance between supporting components.
closed drainage system	A buried system composed of catch basins and culverts used to drain roadways.
closure pour	A placement of cast-in-place concrete used to connect two or more previously cast portions of a structure.
cofferdam	An enclosed cell constructed in the water, generally consisting of sheet piling, from which the water is removed so as to allow construction to take place inside. Also, a partially closed structure made of sandbags or the like, for diverting water away from a shoreline or a structure for the same reason.
column	A vertical support member typically several times taller than its width.
component	A constituent part of a structure.
composite beam	A beam connected to a deck so that they respond to force effects as a unit.
concrete	A mixture of portland cement and stone aggregates combined with water, causing the cement to bind the mass together. Concrete is high in compressive strength but weak in tension or bending. It may contain various admixtures to increase its strength or improve its workability. May be reinforced or unreinforced.
concrete cover	The distance between a concrete face and the nearest main reinforcement bar.
concrete joint	The divisions between individual placements of concrete in a structural unit. Common joint types are construction, contraction, and expansion.
concrete pedestal	A formed platform on a bridge seat used to support a bridge bearing.
connection plate	Secondary member connecting two elements (e.g., a connection plate joining a diaphragm to a girder).
construction joint	A temporary joint used to permit sequential construction.
contraction joint	A joint placed every 30 feet along a wall to control the location of cracks. Without these joints, the concrete would form cracks at unpredictable intervals. Reinforcing steel is normally not carried through the joint except in rigid frame structures, where moment must be transferred from wall to slab.
cover plate	A steel plate attached to the flange of a beam to increase the overall section properties of the member.
creep	Time-dependent deformation of concrete under permanent load.

Structures	
cross-frame	A typically X-shaped combination of members placed transversely to the main beams or girders to distribute stresses and improve strength and rigidity. (See diaphragm.)
culvert	Any structure not defined as a strut or bridge that provides a drainage opening under the roadway or approaches to the roadway.
curtain	A system used with finger joints that protects the girder ends from water coming through the joint. Neoprene curtains are placed at the end of each girder, allowing the water to be dumped diffusely on a protected bridge seat that is periodically cleaned by maintenance forces.
curved girder	A girder curved to accommodate horizontal alignment.
dead load	The permanent self weight of a structure and its components.
deck	The topmost roadway surface of a traditional bridge, supported by the beams or girders and carrying the vehicular and pedestrian traffic.
deck joint	A complete or partial interruption of the deck to accommodate relative movement between portions of a structure.
deck truss	A truss system in which the roadway is at or above the level of the top chord of the truss.
deck width	The fascia-to-fascia width normal to the centerline.
defect	A discontinuity or discontinuities that by nature or accumulated effect render a part or product unable to meet minimum applicable acceptance Standards or Specifications. The term implies rejectability.
delamination	The separation of a lamination under stress.
design span	For decks, the center-to-center distance between the adjacent supporting components, taken in the primary direction.
development length	The distance required to develop the specified strength of a rebar or prestressing strand.
diaphragm	A typically solid member placed transversely to the main beams or girders to distribute stresses and improve strength and rigidity. (See cross-frame.)
disc bearing	A bearing that accommodates rotation by deformation of a single elastomeric disc molded from a urethane compound. It may be movable, guided, unguided, or fixed. Movement is accommodated by sliding of polished stainless steel on PTFE.
distribution slab	A pier footing constructed on top of a cofferdam seal.
drain downspout	The outlet pipe of a bridge drain.

Structures	
drilled shaft	A deep foundation unit, wholly or partially embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacity from the surrounding soil and/or from the soil or rock strata below its tip. Also commonly referred to as caissons, drilled caissons, bored piles, or drilled piers.
elastomer	Any of various polymers with elastic properties resembling those of natural rubber.
elastomeric bearing	A bearing made of elastomer, often reinforced with several layers steel plates.
embedment length	The length of rebar or anchor provided beyond a critical section over which transfer of force between concrete and reinforcement may occur.
expansion device	A fabricated unit installed at a deck joint in a bridge deck to allow for thermal movement between the superstructure and substructure. Some types of expansion devices include a joint seal to prevent drainage of water or debris from getting to the bridge seat. Types of expansion devices include compression seals, gland seals, finger joints, and modular joints.
expansion joint	A joint used to prevent compression forces from abutting concrete from crushing or displacing the adjacent structure. They are designed to isolate one structural element from another, and can occur where expansion forces in elements change direction. On a retaining wall, an expansion joint should be used every 90 feet. Reinforcing steel is not carried through the joint.
extraordinary bridge	A structure with a length of 250 feet or more which has an improvement cost of at least \$5 million.
fascia	The outside face of a concrete deck.
fascia girder	The outside or exterior girder in a set of beams.
fatigue	The initiation and/or propagation of localized cracks due to a repeated variation of normal stress with a tensile component.
fill concrete	Unreinforced concrete used as fill material.
fixed bearing	A bearing that prevents differential longitudinal translation of abutting structural elements. It may or may not provide for differential lateral translation or rotation.
flange	The top and bottom horizontal component of an "I" girder or horizontal component in a "T" section.

Structures	
flatness	The degree of which a surface deviates from a plane. Used often with the term "flatness tolerance". Flatness tolerance shall be described by a unit number x the length or width of a plate. The resultant representing the maximum gap permitted beneath an edge placed parallel to the width or length of a plate.
floor beam	A secondary load-carrying member spanning transversely between trusses or other primary longitudinal members.
flowable fill	A loose, low-strength concrete material. Primarily used to fill gaps, such as that created by a sliplined culvert and the existing structure.
footing	The lowest portion of a substructure unit that distributes the structure loads either to the earth or to supporting piling.
fracture critical	A tension or stress reversal member whose failure would be expected to result in the failure of the structure.
french drain	A groundwater drainage system constructed of stone and placed behind substructure walls.
friction pile	A pile whose support capacity is derived principally from soil resistance mobilized along the side of the embedded pile.
F-shape barrier	A solid, reinforced concrete barrier used as permanent bridge rail, typically on overpass structures.
gravity abutment	An abutment constructed of concrete with front and back batters that relies on its self weight to support its applied loads. A gravity abutment is either lightly reinforced for temperature and shrinkage or is unreinforced. Also called a mass abutment. See Figure A-3.
gusset plate	A steel plate used to connect steel members.
hanger	An assembly utilizing a pin connection to allow for expansion between a cantilevered and suspended span between supports.
haunched girder	A girder that has a deeper web over the pier that is transitioned over a short distance to a shallower constant depth web.
heel	The back protruding portion of a footing for a wall or abutment.
high performance concrete	Concrete that is typically used in precast, prestressed applications designed for a compressive strength equal to or greater than 5000 psi. There are other definitions of HPC, but this is the one commonly used at MaineDOT.
highway culvert	A buried structure with a span of less than 10 feet.

Structures	
hybrid girder	A girder made from plates of different steel types (e.g., high-strength steel used for flanges and lower strength for webs).
I-girder	A precast, prestressed concrete girder whose cross section somewhat resembles the letter "I". No longer used to refer to steel I-girders as they have been replaced with wide flange shapes.
inlet	Typically used to reference the upstream end of a culvert structure or the entrance grate to a catch basin or bridge drain.
integral abutment	An abutment consisting of a breastwall supported directly on driven piles or spread footings and constructed so as to act as a single unit with the superstructure. An integral abutment eliminates the need for an expansion device and therefore the associated potential maintenance problems.
jacking force	The force exerted by the device that introduces tension into tendons.
joint seal	A poured or preformed elastomeric device designed to prevent moisture and debris from penetrating joints.
lateral bracing	Bracing located near the top flange or bottom flange or chord of a beam, girder or truss to prevent lateral deformation induced by forces normal to the bridge centerline (e.g., wind).
live load	The transient loads supported by a structure, usually referring to the vehicle/truck loads and pedestrian loads.
masonry plate	The bottom steel plate of a bearing anchored to a concrete bridge seat.
membrane waterproofing	A hot-applied rubberized sheet applied to the top of a concrete deck prior to installation of the bituminous wearing surface. This system is intended to stop the ingress of salt into the substrate concrete of the deck, thereby preventing concrete delamination and corrosion of reinforcing steel.
metal rocker or roller bearing	A fabricated steel bearing formerly used to support steel beams. May also be called a pedestal bearing.
minor span	A structure having a clear span equal to or greater than 10 feet and less than 20 feet along the centerline of the road, or in case of multiple spans, when the combined clear spans equal or exceed 10 feet and is less than 20 feet.
multi-plate	An alternate designation for structural plate.

Structures	
neat line	A reference line from which dimensions are measured. Typically used on substructure details when the bottom of footing is founded on bedrock and the exact location of the bedrock is uneven or unknown.
non-voided slab	A solid precast, prestressed concrete slab with no circular voids. Non-voided slabs are typically 3 feet or 4 feet wide and are installed as butted beams post-tensioned together.
nose	The protruding portion of a pier shaft facing the oncoming flow of water.
ordinate	The coordinate dimension representing the distance from a specified point to the x-axis, measured parallel to the y-axis.
outlet	Typically used to reference the downstream end of a culvert structure.
pier	The intermediate supporting structures of a traditional bridge. Types of piers included column, hammerhead, cantilevered, mass, and bent. See Figure A-8.
pile	A vertical member driven into the soil to support the substructure units of a traditional bridge. Types of piles include steel H-piles, steel pipe piles, and precast, prestressed concrete piles.
pile bent	A pier consisting of a pier cap supported by exposed piles.
pile shoe	A metal piece fixed to the penetration end of a pile to protect it from damage during driving and to facilitate penetration through very dense material or boulders.
pipe arch	A commonly used bridge culvert with an oblate shaped opening.
plate	A flat rolled product whose thickness exceeds 1/4 inch.
plate girder	A girder with an "I" cross section fabricated from steel plates joined by welding, bolting, or riveting.
point-bearing pile	A pile whose support capacity is derived principally from the foundation material upon which the pile tip rests. Also known as an end-bearing pile.
post-tensioning	A method of prestressing in which the tendons are tensioned after the concrete has reached a predetermined strength.
precast concrete	Concrete components that are cast in a location other than their final position.
prestressed concrete	Concrete components in which stresses and deformations are introduced by the application of prestressing forces.
prestressing strand	High strength steel wires used for tension reinforcement in precast, prestressed beams.

Structures	
rebar	A shortened, alternative designation for reinforcing steel.
redundancy	The quality of a bridge that enables it to perform its design function in a damaged state.
redundant member	A member whose failure does not cause failure of the bridge.
reinforcing steel	A series of deformed steel bars placed in a concrete structure to increase its strength in tension and bending and to prevent cracking due to thermal changes.
residual stress	Stress locked into a member after it has been worked to its final shape.
retaining wall	A structure designed to retain and hold back a mass of earth. Types of retaining walls include cantilevered, gravity or mass, MSE, and PCMG.
return wingwall	A wing that extends back from the abutment and parallel to the roadway alignment.
rigid frame	A bridge structure consisting of a top slab with two integral walls forming an inverted "U" shaped opening. (See traditional bridge and buried structure.)
rolled beam	A steel beam formed by hot-rolling.
scabble	To remove the top portion of existing deck concrete with a machine that pounds the surface.
scarify	To remove the top portion of existing deck concrete with a machine that grinds the surface.
scupper	A drainage system used to drain storm water runoff from a bridge deck.
seal	The concrete placed underwater inside a sheet pile cofferdam to close the bottom and allow the water to be removed. See Figure A-1.
secondary member	Bracing between primary members designed to resist cross-sectional deformation of the superstructure frame and help distribute part of the vertical load between stringers
semi-integral abutment	An abutment that behaves in the conventional manner, while the backwall (end diaphragm) moves along a horizontal joint below ground.
shaft	The main portion of a solid pier, often supporting a separate cap.
shear key	A formed, rectangular notch used between successive placements of concrete.
sheet	A flat rolled product whose thickness is less than 1/4 inch.
sheet piling	A series of interlocking steel shapes driven vertically into the soil to form a wall or enclosed cell. (See cofferdam.)
sidewalk	A way constructed primarily for use by pedestrians.

Structures	
skew or skew angle	The acute angle formed by the intersection of the line normal to the centerline of the roadway or the working line of the superstructure with a line parallel to the face of the substructure or in the case of structural plate units and culverts, with the centerline of the structural plate units and culverts. The skew is always described as "ahead" or "back" on the left side of the centerline of construction.
slab	A component having a width of at least four times its effective depth.
sole plate	The top plate of a bearing attached to the bottom flange of a beam, girder, or box.
spandrel arch	An arch in which the roadway is supported on top of columns (open) or walls (closed) constructed on the arch ribs.
splice	The joining of two elements on the same plane through a connection device. Components such as rebar, piles, and beams are often spliced.
stiffener	A plate welded to a steel beam web to enhance section properties of the beam. Intermediate stiffeners are welded vertically and longitudinal stiffeners along the length of the beam. Bearing stiffeners are welded vertically at bearing locations.
stringer	A secondary load-carrying member spanning longitudinally between transverse floor beams.
structural concrete	Another term for reinforced concrete.
structural plate	A metal structural plate structure used as a minor span or bridge. Types include pipe, pipe arch, plate arch, or box frame.
structural steel	A generic term for steel used in structural applications.
structure	Bridge, culvert, catch basin, drop inlet, retaining wall, cribbing, manhole, endwall, building, sewer, services pipe, underdrain, foundation drain, or other manufactured feature.
strut	Any structure not defined as a culvert, minor span, or bridge that provides a drainage opening under the roadway or approaches to the roadway that is over 5 feet and less than 10 feet in span or nominal diameter.
strut	The transverse member in a lateral bracing system.
stub abutment	A short abutment usually founded on piles. Typically used with fill embankments. See Figure A-5.

Structures	
substructure	All of that part of the structure below the bearings of simple and continuous spans, skewbacks of arches, and tops of footings of rigid frames, plus the backwalls, parapets, and wingwalls of abutments. See Figure A-2.
superimposed dead load	The dead loads that are applied to the design of a composite beam section.
superstructure	The entire traffic-carrying portion of a traditional bridge structure above the bridge seats, including bearings, beams or girders, deck, curbs and bridge railing, but excluding backwalls, wingwalls, and wing protection railing. See Figure A-1.
surface finish	The relative smoothness of a surface usually defined in thousandths of an inch or microns allowing the maximum permissible deviation from a straight line; may be applied to flat or round surfaces.
sweep	Curving of the girder in the horizontal plane.
T girder	A girder whose cross section resembles the letter "T".
temporary concrete barrier	A precast concrete barrier that is typically used to delineate temporary travel lanes for traffic control during construction.
tendon	A high-strength steel element used to prestress concrete.
through girder	A girder system where the roadway is below the top flange.
through truss	A truss system where the roadway is located near the bottom chord and a top chord lateral system is provided.
tied arch	An arch in which the horizontal thrust of the arch rib is resisted by a horizontal tie.
toe	The front protruding portion of a footing for a wall or abutment.
traditional bridge	A bridge structure consisting of separate substructure and superstructure units. (See buried structure and rigid frame.)
transverse	The direction perpendicular to the length of a bridge and the centerline of construction.
traveled way	The portion of the roadway that is intended for the movement of vehicles, exclusive of shoulders and auxiliary lanes.
trough	A system used with finger joints that collects and directs the water coming through the joint via a drain to a specific location.
true arch	An arch in which the horizontal thrust of the arch rib is resisted by an external force supplied by its foundation.

Structures	
truss	In a traditional bridge, a main load-carrying member spanning longitudinally between supports, consisting of individual components assembled into a rigid triangular framework.
turn-of-wing	The angle point between the abutment and the wingwall, where the wingwall changes plane and begins to slope downward. For a typical non-curbed approach roadway, the “turn-of-wing” is located at the shoulder berm.
viaduct	A structure of some length carrying a state highway over streets, railroads, or other various features.
voided slab	A precast, prestressed concrete slab with circular voids. Voided slabs are typically 3 feet or 4 feet wide and are installed as butted beams laterally post-tensioned together.
wearing surface	A layer of concrete, asphalt pavement or other material placed over a bridge deck to protect the concrete.
web	The vertical component of a girder or beam.
welded girder	In a traditional bridge, a main load-carrying member spanning longitudinally between supports and consisting of several individual pieces welded into one built-up section. (See beam.)
wing	A shortened alternative name for wingwall.
wingwall	The retaining wall extension of an abutment, serving to retain the sides of the approach roadway embankment.
wingwall parapet	The top surface of an abutment between the bridge curb and the wingwall. The parapet allows for the change in plane of the top of a sloping wingwall. A parapet is constructed so that its top surface is level.
yield point	The stress at which a material permanently deforms.

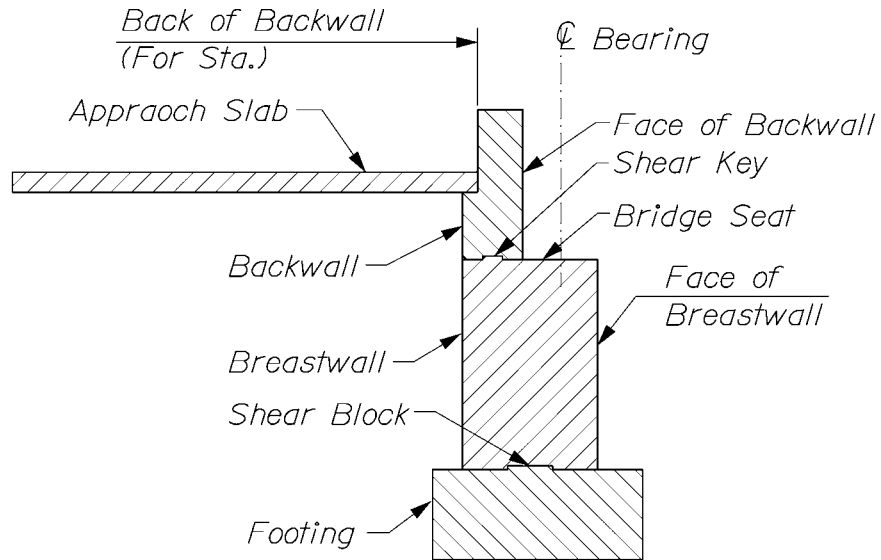
A.1.8 Welds

Welds	
ASTM A6	Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling. Describes the dimensional tolerances, allowable imperfections, and methods of conditioning (repairing) the steel.
base metal	Material that is welded, heated, or cut.
complete joint penetration	Penetration of weld metal through the entire thickness of a joint with a groove weld (used only when needed to develop the full capacity of a member).
couplant	A substance used to ensure intimate contact of equipment with steel when performing UT (Ultrasound Testing).
coupon	A sample taken from a larger element used to determine the mechanical, physical, and/or chemical properties of that element. A test specimen.
discontinuity	Imperfections in welds or base metals. A discontinuity is not necessarily a defect. A discontinuity becomes a defect when it exceeds acceptable limits in accordance with the applicable Codes and Standards.
electrode	A component of the welding circuit through which current is conducted between the electrode holder and the arc. An electrode may or may not provide filler metal.
ESW weld process	Electro Slag Welding. A welding process that produces coalescence of metals with molten slag that melts the filler metal and the surfaces of the workpieces. The process is initiated by an arc that heats the slag. The arc is then extinguished by the conductive slag, which is kept molten by its resistance to electric current passing between the electrode and the workpieces.
FCAW weld process	Flux Core Arc Welding. A welding process that uses an arc between a continuous filler metal electrode and the weld pool.
FCAW-G weld process	Flux Cored Arc Welding (gas-shielded). A flux cored arc welding process variation in which shielding gas is supplied through the gas nozzle, in addition to that obtained from the flux within the electrode.
FCAW-S weld process	Flux Cored Arc Welding (self shielded). A flux cored arc welding process variation in which shielding gas is obtained exclusively from the flux within the electrode.

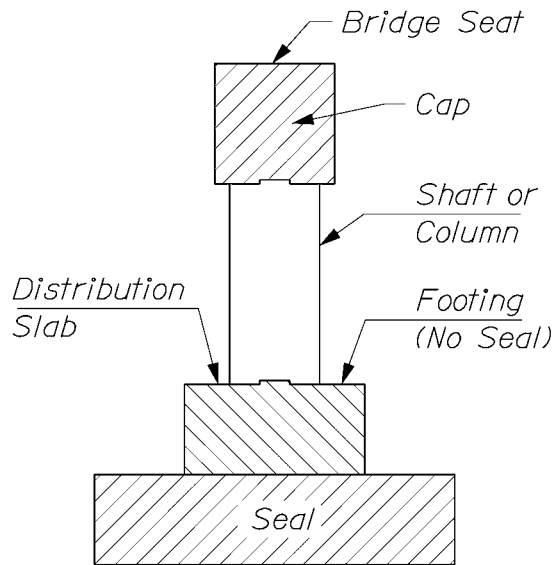
Welds	
globular weld process	The metal droplets are much larger in size, irregular in shape and short circuits occur at irregular intervals. Occurs when the current density is lower than that required for spray transfer or when CO ₂ , or mixtures of argon and CO ₂ are used.
GMAW weld process	Gas Metal Arc Welding. An arc welding process that uses a continuous consumable electrode and a shielding gas.
GTAW weld process	Gas Tungsten Arc Welding. An arc welding process that uses an arc between a tungsten electrode (non-consumable) and the weld pool. This process is used with shielding gas and without the application of pressure.
heat input	The ratio of the arc power entering the workpiece to the weld travel speed. Excessive heat input often reduces weld metal and heat affected zone strength and toughness. Low heat input can increase the incidence of fusion defects and affect soundness and mechanical properties.
lamellar tearing	A fracture separation in heavy weldments, found within or just beneath the heat affected zone of thick weldments.
lamination	Flat, generally elongated, planar base metal discontinuities found near the center of rolled products. They generally run parallel to the surface of the rolled product and are most commonly found in structural shapes and plates.
partial joint penetration	Groove welds without steel backing, welded from one side, groove welds welded from both sides but without backgouging.
Procedure Qualification Record	A record of welding variables used to produce an acceptable test weldment and the results of tests conducted on the weldment to qualify a Welding Procedure Specification.
pulsed arc weld process	Spray transfer occurs in pulses at regularly spaced intervals rather than at random intervals. In the time between pulses, the welding current is reduced and no metal transfer occurs.
SAW weld process	Submerged Arc Welding. A welding process that uses an arc or arcs between a bare metal electrode or electrodes and the weld pool. The arc and molten metal are shielded by a blanket of granular flux on the workpieces.
seal weld	Any weld designed primarily to provide a specific degree of tightness against leakage.

Welds	
short circuit weld process	Occurs with low current density and low voltage. The arc short-circuits up to 200 times per second, and the metal is transferred during the short circuit. Used on thin sheets and is especially applicable to vertical and overhead welds when fit-up is poor.
SMAW weld process	Shielded metal arc welding (stick welding). A welding process with an arc between a covered, consumable electrode and the weld pool.
spray weld process	Very fine droplets of metal being transferred in the arc and a high current density on the electrode. The shielding gas is either argon or a mixture of argon and oxygen. No short circuits occur in the arc.
SW weld process	Stud Arc Welding. An arc welding process that uses an arc between a metal stud, or similar part, and the other workpiece. The process is used with or without shielding gas or flux, with or without partial shielding from a ceramic ferrule surrounding the stud, with the application of pressure after the faying surfaces are sufficiently heated, and without filler metal.
transfer mode weld process	The manner in which the molten metal travels from the end of a consumable electrode across the welding arc to the workpiece. Only spray and globular transfer modes are permitted on bridges.
ultrasonic gage	A device used to measure the thickness of steel or for locating discontinuities using a transducer and CRT (Cathode Ray Tube).
weld	The joining of two metal parts by applying heat, sometimes with pressure. Typical types of weld include fillet and groove, and they may have full or partial penetration.
weld soundness	The overall quality of a weld that establishes fusion to base metal and between weld passes.
Welding Procedure Specification	A document providing, in detail, the required variables for specific application to assure repeatability by properly trained welders and welding operators.

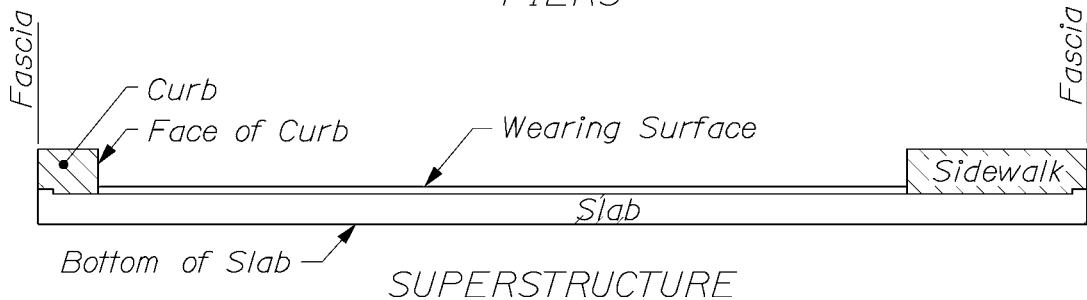
A.2 Drawings



ABUTMENTS

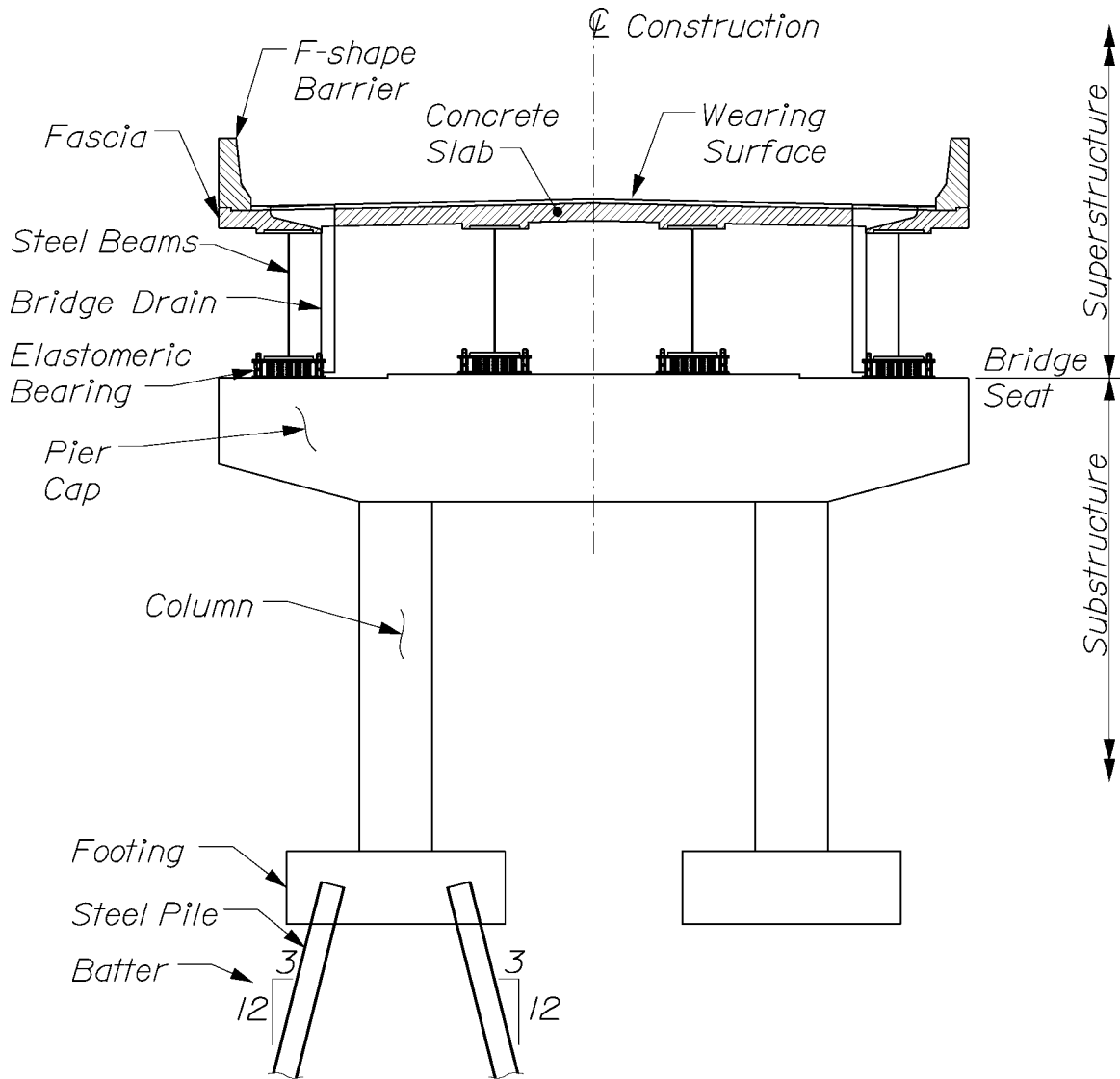


PIERS



SUPERSTRUCTURE

Figure A-1 Abutment, Pier, Superstructure



TRANSVERSE SECTION

Figure A-2 Superstructure and Substructure

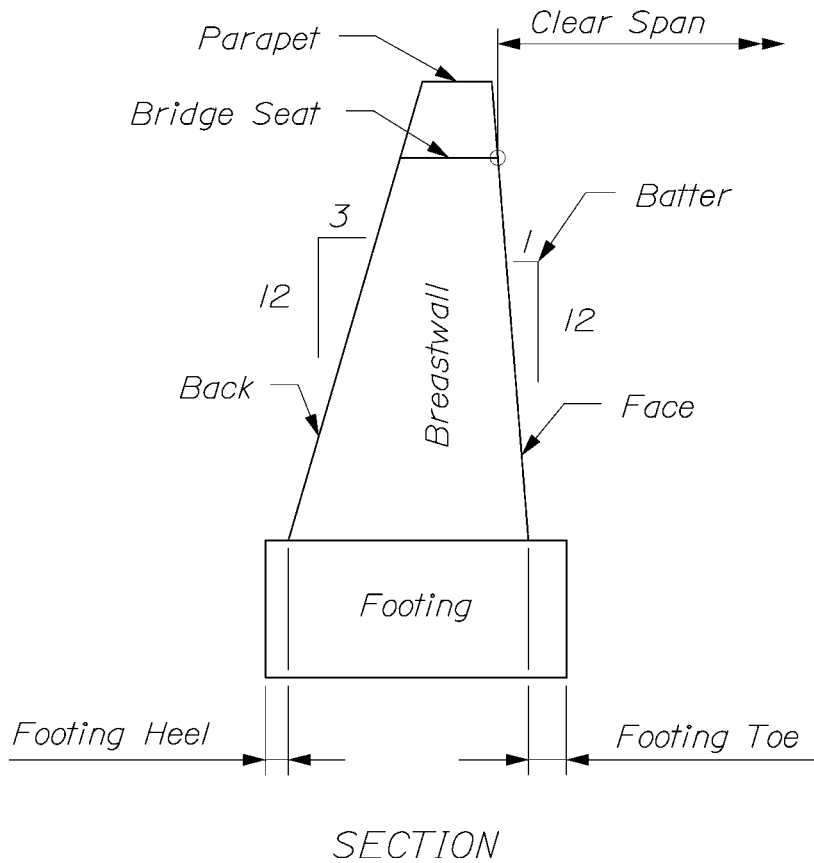
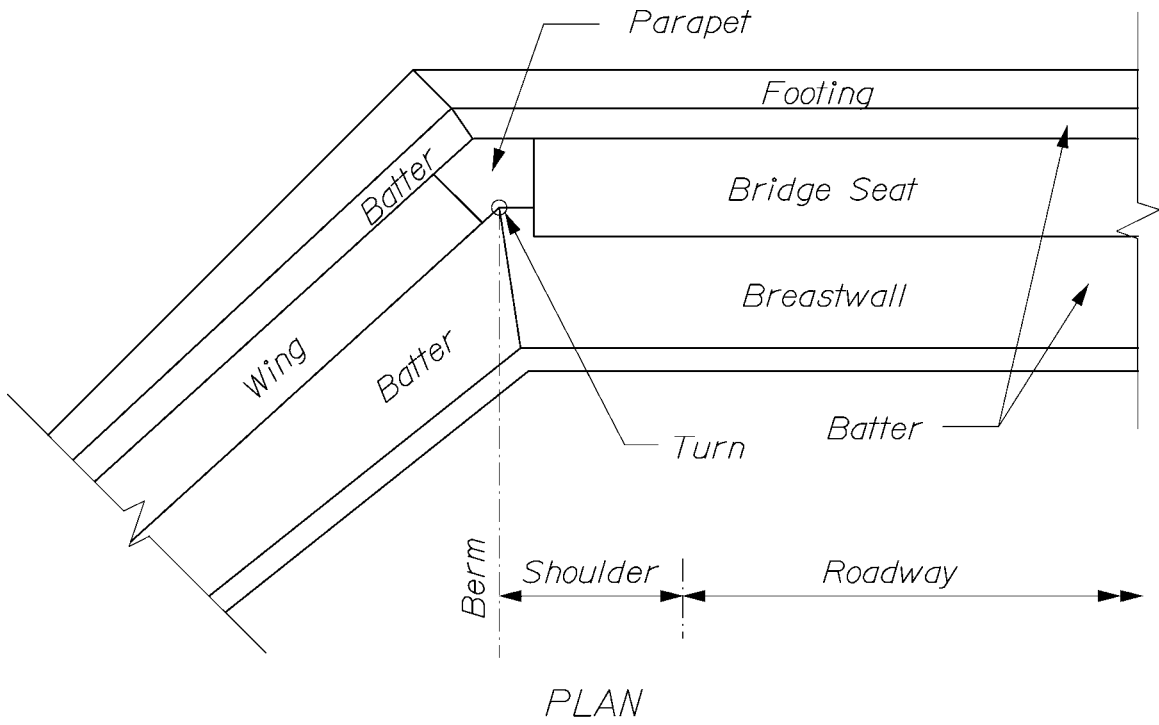


Figure A-3 Gravity Abutment

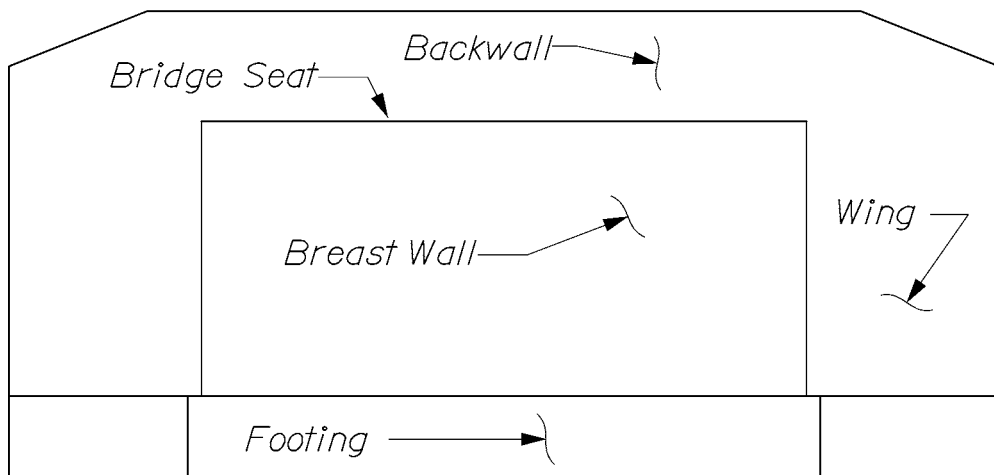
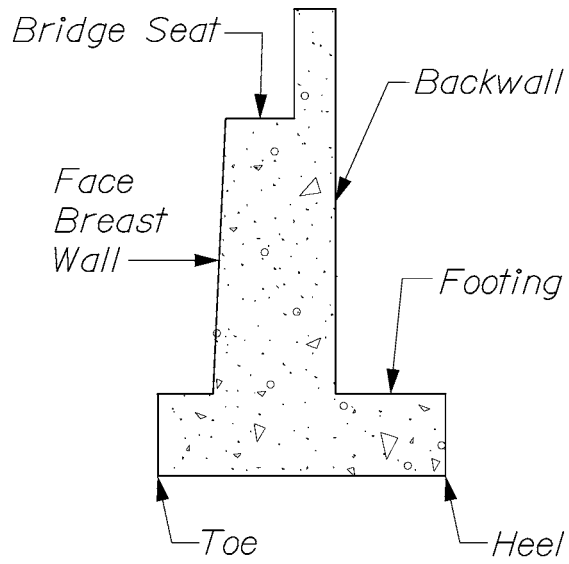


Figure A-4 Full Height Cantilever Abutment

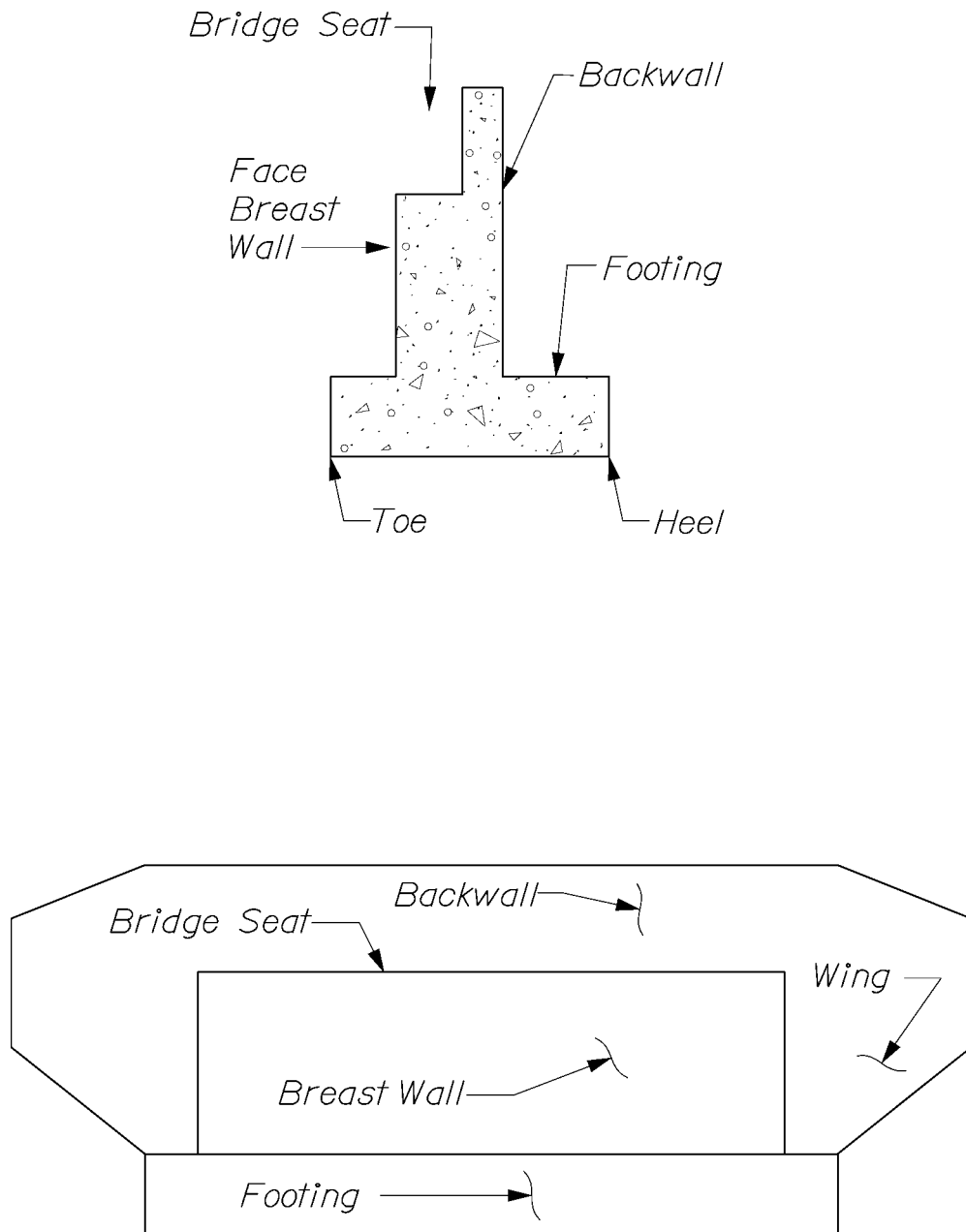


Figure A-5 Stub Abutment

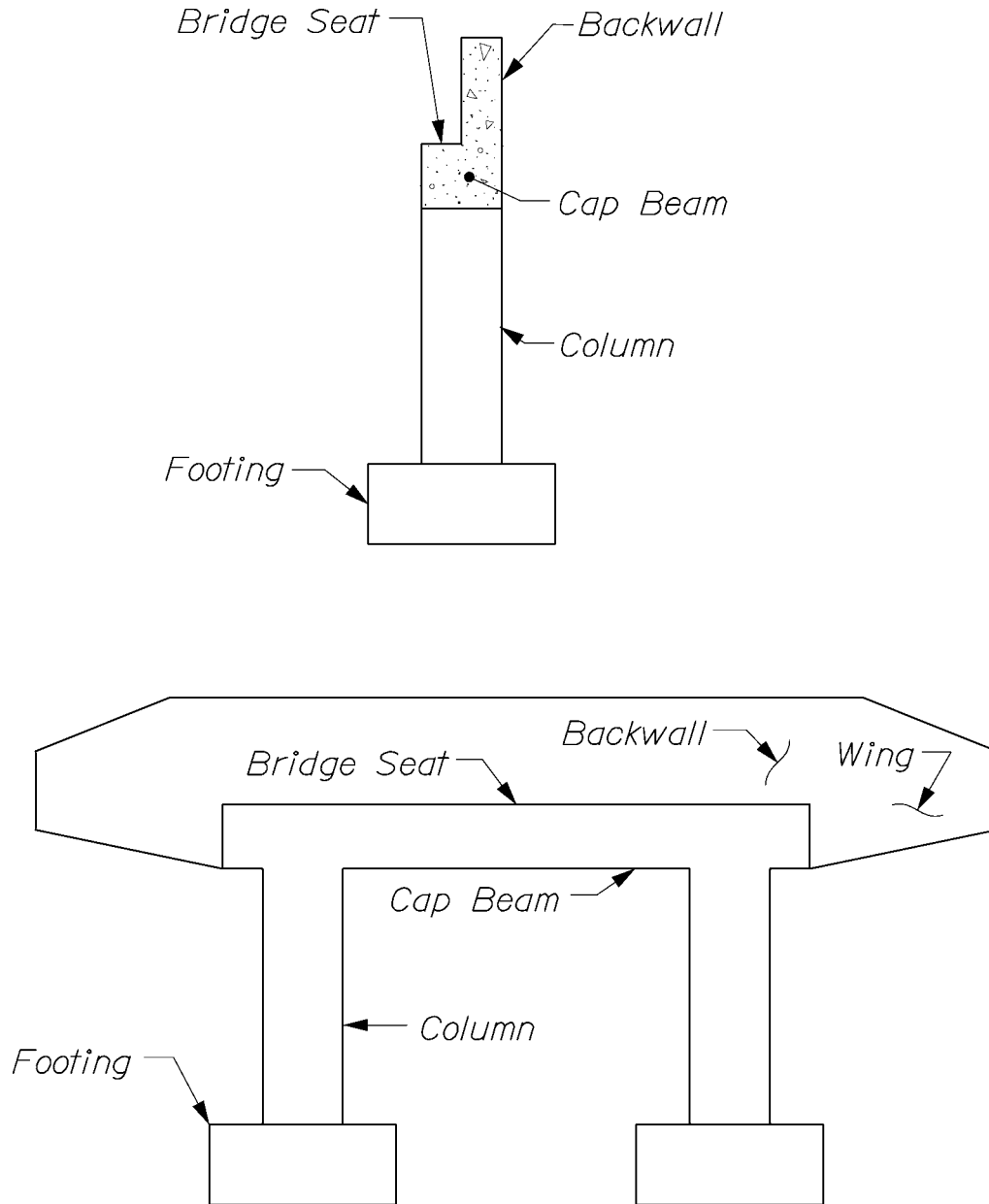


Figure A-6 Open Abutment

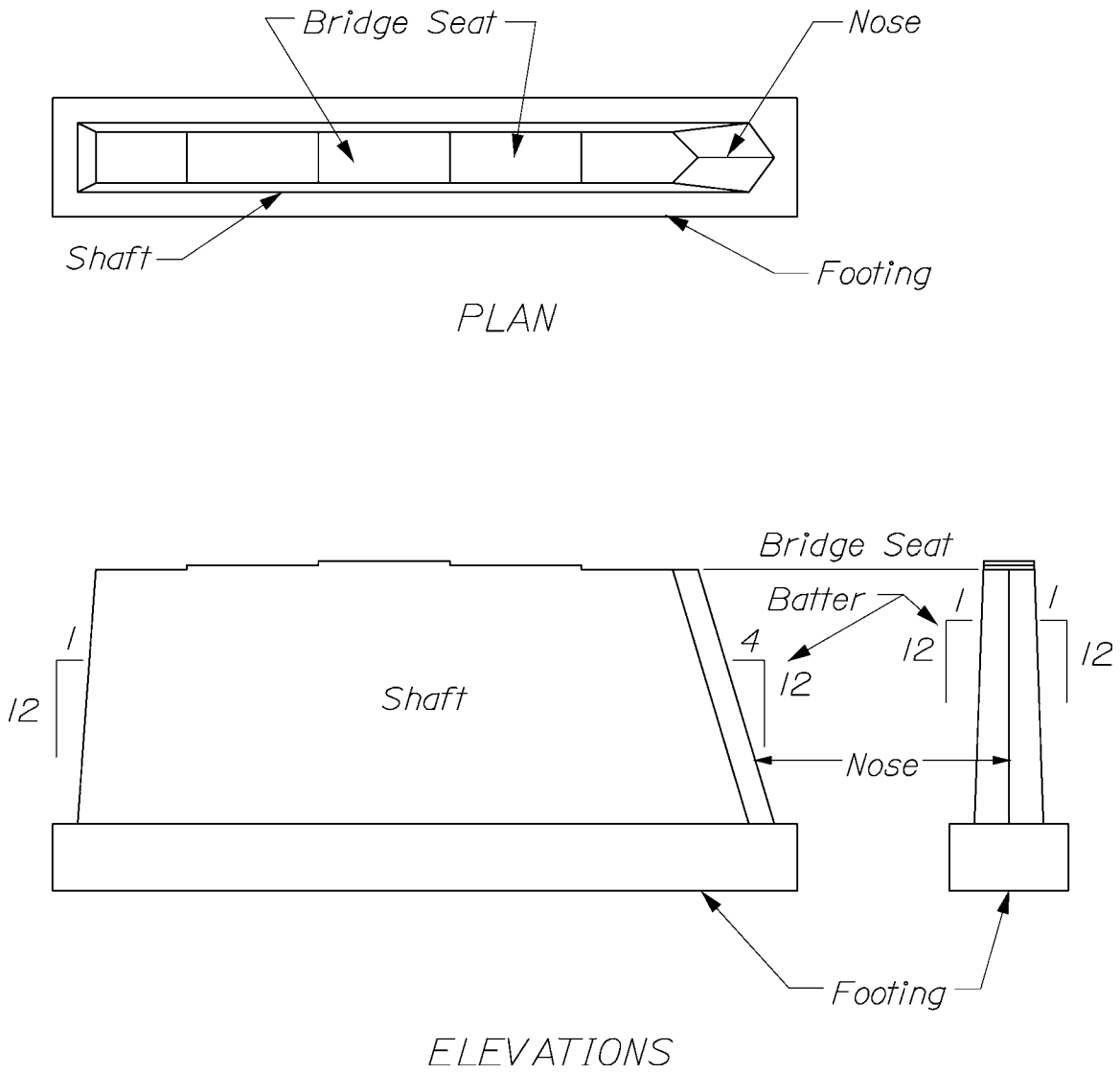


Figure A-7 Solid Pier

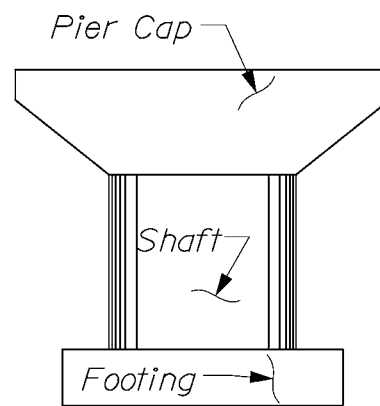
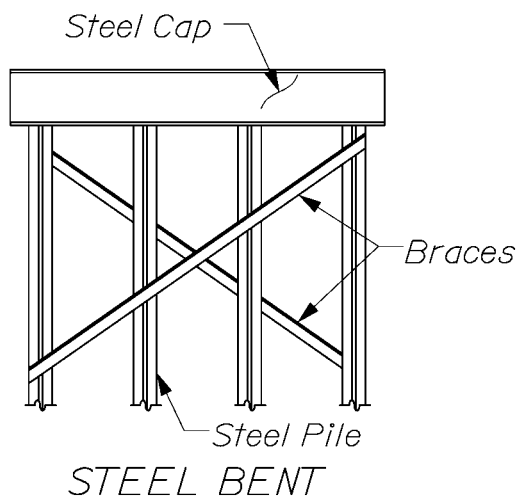
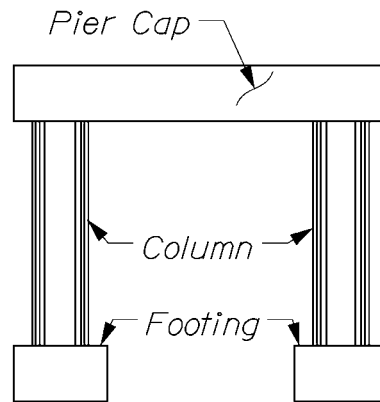
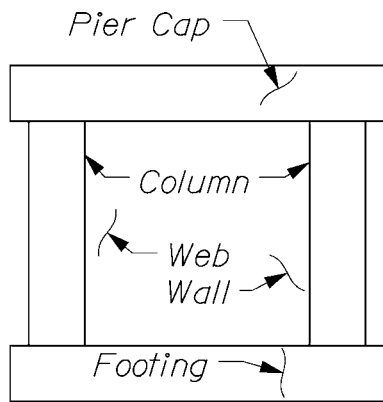
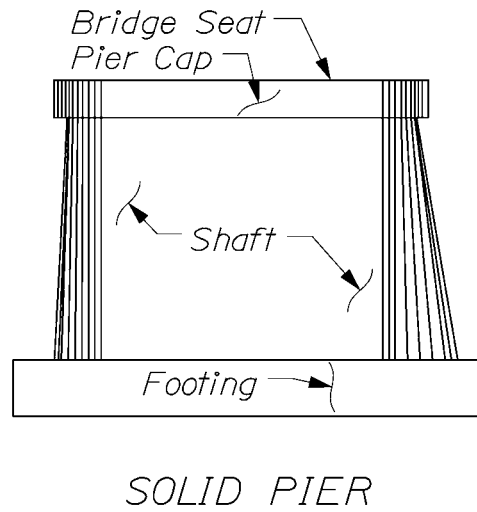
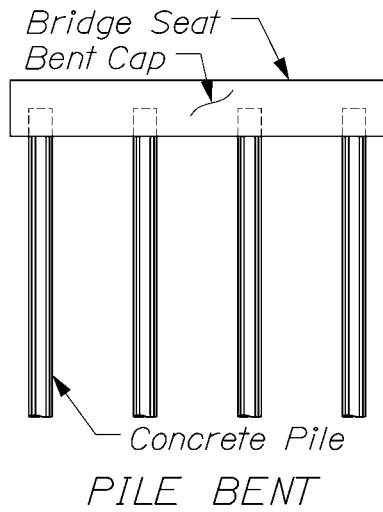


Figure A-8 Types of Piers

Appendix B

PDR FORMS



Wire Bridge, New Portland



Donald V. Carter Bridge, Waterville-Winslow

Appendix B PDR Forms (Section Removed)

The PDR form examples have been removed from the Bridge Design Guide and are available as electronic Microsoft Word files instead.

Appendix C

HYDROLOGY/HYDRAULICS



Scotty Richardson Bridge, Rumford



Covered Bridge, Norridgewock

Appendix C Hydrology/Hydraulics

C.1 Transposed Discharge Methods

The following three transposed discharge methods are used for ungaged sites on gaged, unregulated streams in rural drainage basins that are between 50% and 200% of the drainage area of the gaging station, except for sites that are plus or minus 3% of the drainage area. Ungaged sites within 3% of the gaging station drainage area, should use the values in "Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals by Glen Hodgkins 1999, U.S. Geological Survey Water -Resources Investigations Report 99-4008 Table 1 pages 8 -17. Please note that Q_{50} is the same as the Q50 designation used elsewhere in this Guide – the same is true for other flow rates as well.

C.1.1 Transposed Discharge Method #1

This method is for calculating a final weighted peak flow at an ungaged site on a gaged stream by weighting the peak flow from the gaging station with the peak flow from the U.S.G.S. Hodgkins full regression equation.

$$Q_{uf} = Q_r(W_r) + Q_u(1 - W_r)$$

in which:

Q_{uf} - the final weighted peak flow for a given recurrence interval (for example, Q_{50}) for an ungaged site on a gaged stream.

Q_r - the regression estimate of the peak flow, at the ungaged site, for a given recurrence interval (for example, Q_{50}) using the U.S.G.S. Hodgkins full regression equation

W_r is a weighting factor:

$$\begin{aligned} \text{For } A_u > A_g, W_r &= (A_u/A_g) - 1 \\ \text{For } A_u < A_g, W_r &= (A_g/A_u) - 1 \end{aligned}$$

in which:

A_u is the drainage-basin area of the ungaged site
 A_g is the drainage-basin area of the gaging station

Q_u is the peak flow for the gaging station with a drainage area adjustment

$$Q_u = Q_w(A_u/A_g)^b$$

in which:

APPENDICES

Q_w is the weight-average peak flow for a given recurrence interval (such as Q_{50}) for the gaging station from table 1 in “Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals” by Glen Hodgkins, U.S. Geological Survey Water-Resources Investigation Report 99-4008.

b is the coefficient of the simplified (drainage area only) regression equation for the appropriate recurrence interval:

- b = 0.825 for a recurrence interval of 2-years
- b = 0.797 for a recurrence interval of 5-years
- b = 0.783 for a recurrence interval of 10-years
- b = 0.767 for a recurrence interval of 25-years
- b = 0.757 for a recurrence interval of 50-years
- b = 0.748 for a recurrence interval of 100-years
- b = 0.729 for a recurrence interval of 500-years

C.1.2 Transposed Discharge Method #2

If the explanatory variable (drainage area and percentage of basin wetlands) are:

- a. outside the 2-dimensional range of the variables used for the U.S.G.S.Hodgkins Full Regression Equation figure 1 or
- b. if the ungaged site had Canadian Drainage, then

$$Q_{uf} = Q_w(A_u/A_g)^b$$

in which:

Q_{uf} is the final weighted peak flow for a given recurrence interval (for example, Q_{50}) for an ungaged site on a gaged stream

Q_w is the weighted-average peak flow for a given recurrence interval (such as Q_{50}) for the gaging station from table 1 in “Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals” by Glen Hodgkins, U.S. Geological Survey Water-Resources Investigation Report 99-4008 (or from future reports). If the weighted-average flow is not available, the gaging-station peak flow should be used.

C.1.3 Transposed Discharge Method #3

This method is for determining flows at an ungaged site located between two gaging stations.

$$Q_{uff} = (Q_{uf1}(A_{g2} - A_u) + Q_{uf2}(A_u - A_{g1}))/ (A_{g1} - A_{g2})$$

in which:

APPENDICES

Q_{uff} is the weighted flow for an ungaged site between gaging station 1 & 2

Q_{uf1} is from transposed method #1 or transposed method #2 (as appropriate) for the upstream gaging station

A_{g2} is the drainage-basin area of the downstream gaging station

A_u is the drainage-basin area of the ungaged site

Q_{uf2} is from transposed method #1 or transposed method #2 (as appropriate) for the downstream gaging station

A_{g1} is the drainage-basin area of the upstream gaging station

C.2 Discharge Adjustment Factors

The following table can be used to estimate flows, based upon a known Q_{50} . These factors should not be used to adjust flows provided by the MaineDOT Hydrology Section based on the U.S.G.S. equations.

Table C-1 Discharge Adjustment Factors

Recurrence Interval (years)	Factor to Apply to Q50
1.1	.32
2	.42
5	.56
10	.67
20	.80
25	.85
50	1.0
70	1.07
100	1.18

C.3 Forms

The following forms can be used to assist in gathering the needed data.

REPORT FROM BRIDGE MAINTENANCE SUPERVISOR

TOWN: _____

BRIDGE NO. & NAME: _____

CLEAR SPAN: _____

CLEARANCE TO WATER (upstream side): _____

AVERAGE DEPTH OF WATER (upstream side): _____

SPAN AND RISE DOWNSTREAM: _____
(if significantly different from above)

EVIDENCE OF SCOUR: none _____ slight _____ much _____

EVIDENCE OF LENGTH:
unlikely _____ likely _____ positive _____ uncertain _____

ADEQUACY OF OPENING:
undersized _____ appropriate _____ oversized _____ uncertain _____

HIGH WATER INFORMATION

Has water been over the road? _____ Date (if known) _____

Estimated Depth over the road: _____

Reliability of Estimate: poor _____ fair _____ good _____

Highest known water elevation relative to bottom of bridge: _____
Date (if known) _____

Was high water caused by a downstream constriction:
no _____ likely _____ positive _____ uncertain _____

Was high water caused by ice:
no _____ likely _____ positive _____ uncertain _____

GENERAL COMMENTS: _____

COMMENTS PERTAINING TO UPSTREAM OR DOWNSTREAM BRIDGES:

DEGREE OF FAMILIARITY WITH SUBJECT BRIDGE:
negligible _____ some _____ considerable _____

Reported by: _____ Date: _____

INFORMATION FROM LOCAL RESIDENT

NAME: _____

ADDRESS: _____

PHONE: _____

YEARS OF RESIDENCE: _____

ADEQUACY OF OPENING: undersized ___ appropriate ___ oversized ___

HIGH WATER RELATIVE TO ROADWAY

Has water been over the road? _____

Estimated depth over the road (if applicable): _____

Flow over the road (velocity): _____

Other information: _____

HIGH WATER RELATIVE TO BRIDGE

Distance from bottom of bridge: _____ above or below

Date of high water: _____

Cause of flood: _____

(ice, spring runoff, hurricane, cloudburst, heavy rain, backwater, debris)

Frequency of flooding:

none ___ seldom ___ occasional ___ frequent ___

Other information:

GENERAL COMMENTS:

COMMENTS PERTAINING TO UPSTREAM OR DOWNSTREAM BRIDGES:

Reported by: _____ **Date:** _____

Appendix D

STANDARD NOTES



Mill Creek Bridge, Falmouth



Bingham, Mill Bridge

Appendix D Standard Notes

The notes on the following pages should be used on the plans where they apply.

D.1 Title Sheet

These notes should appear on the title sheet of the plans, or if a title sheet is omitted, on the general plan.

SPECIFICATIONS

DESIGN: Load and Resistance Factor Design per AASHTO LRFD Bridge Design Specifications Fourth Edition 2007 and interim specifications through 200X.

||
||
||

TRAFFIC DATA

Current (200X) AADT = XXXX
Future (20XX) AADT = XXXX
DHV - % of AADT = XX %
Design Hour Volume = XXX
Heavy Trucks (% of AADT) = XX %
Heavy Trucks (% of DHV) = XX %
Directional Distribution (% of DHV) = XX %
18 Kip Equivalent P 2.0 = XX
18 Kip Equivalent P 2.5 = XX
Design Speed = XX mph

DESIGN LOADING

LIVE LOAD: HL-93 Modified

MATERIALS

CONCRETE:	Structural Wearing Surface	Class LP
	Barriers, Curbs, Sidewalks, End Posts	Class LP
	Seals	Class S
	Precast	Class P
	Fill	Fill
	All Other	Class A

REINFORCING STEEL: ASTM A615, Grade 60

|

APPENDICES

PRESTRESSING STRANDS: AASHTO 203 (ASTM A416), Grade 270, Low Relaxation

STRUCTURAL STEEL:

All Material (unless otherwise noted) ASTM A709, Grade 50W (unpainted)
High Strength Bolts ASTM A325, Type 3

BASIC DESIGN STRESSES

CONCRETE: $f'_c = 4,350$ psi

PRECAST CONCRETE: $f'_c = XX$ psi
 $f'_{ci} = XX$ psi

REINFORCING STEEL: $f_y = 60,000$ psi

PRESTRESSING STRANDS: $f_u = 270,000$ psi

STRUCTURAL STEEL: ASTM A709, Grade 50W $F_y = 50,000$ psi
 ASTM A709, Grade 36 $F_y = 36,000$ psi
 ASTM A325 $F_u = 120,000$ psi

HYDROLOGIC DATA

- Drainage Area = _____ sq mi
- Design Discharge (Q50) = _____ cfs
- Check Discharge (Q100) = _____ cfs
- Headwater Elev. (Q50) = _____ ft
- Headwater Elev. (Q100) = _____ ft
- Discharge Velocity (Q50) = _____ fps
- Discharge Velocity (Q100) = _____ fps
- Headwater Elev. (Q1.1) = _____ ft
- Discharge Velocity (Q1.1) = _____ fps
- Headwater Elev. (Q25) = _____ ft
- Mean Lower Low Water (MLLW) = -X.XX ft
- Mean Low Water (MLW) = -X.XX ft
- Mean Tide Level (MTL) = X.XX ft
- Mean High Water (MHW) = X.XX ft
- Mean Higher High Water (MHHW) = X.XX ft
- 20__ Predicted High Tide = X.XX ft

(The following note is used only when a Coast Guard Permit is required, and should be the only note to be put on the plans in reference to permits.)

COAST GUARD PERMIT REQUIRED

D.2 General Construction Notes

1. All utility facilities shall be adjusted by the respective utilities unless otherwise noted.
2. For easements, construction limits, and right-of-way lines, refer to Right-of-Way Map.
3. During construction, the road will be closed to traffic for a time period specified in the Special Provisions.
4. Place a 24 inch wide strip of Temporary Erosion Control Blanket on the side slopes along the top of the riprap and behind the wingwalls.
5. All embankment material, except as otherwise shown, placed below Elevation XX, shall be Granular Borrow meeting the requirements of Subsection 703.19, Material for Underwater Backfill. ||

(The following note is used when the quantity of clearing is 0.5 acre or less and is to be incidental to contract items.) ||

6. The clearing limits as shown on the plans are approximate. The exact limits will be established in the field by the Resident. Payment for clearing will be considered incidental to related Contract items. ||

(The following note is used when the clearing quantity is more than 0.5 acre and a pay item for clearing is to be included.) ||

7. The clearing limits as shown on the plans are approximate. The actual clearing limits for payment will be established in the field by the Resident. ||
8. Place loam 2 inches deep on all new or reconstructed sideslopes or as directed by the Resident. ||
9. Do not excavate for Aggregate Subbase Course where existing material is suitable as determined by the Resident.
10. In areas where the Resident directs the Contractor not to excavate to the subgrade line shown on the plans, payment for removing existing pavement, grubbing, shaping, ditching, and compacting the existing subbase and layers of new subbase 6 inches or less thick will be made under appropriate equipment rental items.

(The following note is used when unscreened gravel such as aggregate subbase gravel is designated as surface material in the shoulders.)

11. Stones which cannot be rolled or compacted into the surface of the shoulder shall be removed by hand raking. Payment for hand raking will be considered incidental to Item 304.10, Aggregate Subbase Course - Gravel. ||

APPENDICES

12. Deleted. |

(Use the following note when the future AADT is greater than or equal to 500.) |||

13. An NCHRP350 compliant guardrail end treatment shall be installed concurrently with the placement of each section of beam guardrail. |||

(Use the following note when the future AADT is less than 500.) |||

14. A Low Volume Guardrail End shall be installed concurrently with the placement of each section of beam guardrail. |||

(The following note regarding Cable Guardrail is no longer used and has been deleted.) ||

14. Deleted. ||

15. Extended-use Erosion Control Blanket, seeded gutters, riprap downspouts, and other gutters lined with Stone Ditch Protection shall be constructed after paving and shoulder work is completed, where it is apparent that runoff will cause continual erosion. Payment will be made under the appropriate Contract items. |||

(The following note is used for Reduced Berm Offsets.)

16. Guardrail posts as shown in the Standard Details shall be modified from the indicated length of 6 feet to a length of 7 feet, with an embedment of 4.5 feet. Payment will be considered incidental to the guardrail pay items. |||

17. Protective Coating for Concrete Surfaces shall be applied to the following areas: |||

- All exposed surfaces of concrete curbs and sidewalks,
 - Fascia down to drip notch,
 - All exposed surfaces of Concrete Transition Barriers,
 - Concrete wearing surfaces,
 - Concrete barrier railing,
 - Top of abutment backwalls and to one foot below the top of backwalls on the back side.
- |||

18. Erosion Control Mix may be substituted in those areas normally receiving loam and seed as directed by the Resident. Placement shall be in accordance with Standard Specification 619, Mulch. Payment will be made under Item 619.1401, Erosion Control Mix. |

(The following two notes are used in conjunction with Standard Detail 610(2-4).)

19. Place riprap on sideslopes up to elevation XX.

20. Construct the riprap shelf at each abutment at elevation XX.

APPENDICES

(The following five notes are used as needed.)

21. Bidders and Contractors may obtain a copy of the existing bridge plans by faxing a Request for Information to the Bid Contact Person. The plans are reproductions of the original drawings as prepared for the construction of the bridge. It is very unlikely that the plans will show any construction field changes or any alterations, which may have been made to the bridge during its life span.
22. Bidders and Contractors may obtain a copy of the hydrologic report of the bridge site by faxing a Request for Information to the Bid Contact Person. The hydrologic report is based on the Department's interpretation of information obtained for the subject site. No assurance is given that the information or the conclusions of the report will be representative of actual conditions at the time of construction.
23. Bidders and Contractors may obtain a copy of the bridge deck evaluation report of the existing bridge by faxing a Request for Information to the Bid Contact Person. The report contains visual inspection information and deck core data of the bridge. There is no assurance that the information or data is a true representation of the conditions of the entire deck.
24. Bidders and Contractors may obtain a copy of the project geotechnical report(s), Name of Report(s), MDOT Soils Report Number(s), date(s), by faxing a Request for Information to the Bid Contact Person.
25. Geotechnical Information furnished or referred to in this plan set is for the use of the Bidders and the Contractor. No assurance is given that the information or interpretations will be representative of actual subsurface conditions at the time of construction. MaineDOT shall not be responsible for the Bidder's and Contractor's interpretations of, or conclusions drawn from, the Geotechnical Information. The boring logs contained in the plan set present factual and interpretive subsurface information collected at discrete locations. Data provided may not be representative of the subsurface conditions between boring locations.

(The following note is to be used when removing an existing aluminum bridge rail. The Designer should check with Bridge Maintenance to verify the need for this note.)

26. All aluminum bridge rail, rail posts, and associated hardware which are to be removed shall be carefully salvaged by the Contractor and will remain the property of the Department. Payment will be considered incidental to related Contract items.

(The following note is to be used whenever Lump Sum items are included in the Contract.)

27. Quantities included for pay items measured and paid for by Lump Sum are estimated quantities and are provided by MaineDOT for informational purposes only. Lump Sum pay items will be paid for at the Contract Bid

amount, with no addition or reduction in payment to the Contractor if the actual final quantities are different from the MaineDOT provided estimated quantities, except as follows:

- a. If a Lump Sum pay item is eliminated, the requirements of Standard Specifications Section 109.2, Elimination of Items, will take precedence.
- b. If other Contract Documents specifically allow a change in payment for a Lump Sum pay item, those requirements will be followed.
- c. If a design change results in changes to estimated quantities for Lump Sum pay items, price adjustments will be made in accordance with Standard Specifications Section 109.7, Equitable Adjustments to Compensation.

(Include a cofferdam pay item when a cofferdam is clearly required, and identify the location of the cofferdam in the pay item description. When the need for a cofferdam is in question, no pay item will be included, and the following note shall be used. Should both situations arise in the same Contract, the note shall be modified to clearly identify for which cofferdam payment is incidental.)

28. All costs for cofferdams, including pumping, maintenance, related temporary soil erosion and water pollution controls and removal, will not be paid for directly, but will be considered incidental to related Contract items.

(The following note is to be used when a demolition plan is necessary for removal of the existing bridge. This includes trusses, large arch bridges, suspension bridges, bridges that are very close to other structures that are to remain in place, etc.)

29. The Contractor shall submit a Bridge Demolition Plan to the Resident at least 10 business days prior to the start of demolition work. The plan shall outline the methods and equipment to be used to remove and dispose of all materials included in the existing bridge. No work related to the removal of the bridge shall be undertaken by the Contractor until MaineDOT has reviewed the Bridge Demolition Plan for appropriateness and completeness. Payment for all work necessary for developing, submitting and finalizing the Demolition Plan will be considered incidental to the bridge removal pay item.

(The following note is to be used when the existing bridge to be removed contains lead-based paint. Modify the note for rehabilitation projects as necessary.)

30. The existing bridge shall be removed by and become the property of the Contractor. The steel portions of the existing bridge are coated with a lead-based paint system. The Contractor is responsible for the containment, proper management and disposal of all lead-contaminated hazardous waste generated by the process of demolishing the bridge.

APPENDICES

The Contractor is responsible for implementing appropriate OSHA mandated personal protection standards related to this process. Once the existing bridge is removed, the Contractor is solely responsible for the care, custody and control of the components of the existing bridge and any hazardous waste generated as a result of the storage, recycling or disposal of the bridge components, including lead-coated steel. The Contractor shall recycle or reuse the steel in accordance with the Maine Department of Environmental Protection's "Maine Hazardous Waste Management Regulations," Chapter 850. A copy of this regulation is available at MaineDOT's offices on Child Street in Augusta. Payment for all labor, materials, equipment and other costs required to remove and dispose of the existing bridge will be considered incidental to the bridge removal pay item.

D.3 Standard Notes Abutments

1. Reinforcing steel shall have a minimum concrete cover of 2 inches in the walls and 3 inches cover in the footings unless otherwise noted. |||
 2. Cover joints where waterstops are not required in accordance with Standard Detail 502 (01). |||
 3. Place 4 inch diameter drains in breastwall and wingwalls at XX feet maximum spacing. The exact location will be determined by the Resident. |||
 4. Construct French Drains behind the abutments and wingwalls in accordance with Standard Specification Section 512, French Drains. |||
 5. Structural Earth Excavation, Abutments and Retaining Walls, required more than 12 inches below the bottom of the structure, will be paid for in accordance with Standard Specifications Section 206, Structural Excavation. |||
 6. Abutments, wingwalls, and their footings shall be backfilled with Granular Borrow. Pay limits will be the structural excavation limits in cut areas and a vertical plane located 10 feet behind the walls in fill areas. |||
 7. The maximum factored applied footing pressure is XX ksf. |||
- (The following note is used with butted precast box beam or slab type superstructures.)* |||
8. To ensure an accurate match with the superstructure, the parapet portions of the wingwalls shall be placed after erection of the precast units. |||
- (The following two notes are used when Transition Barriers are constructed on return wingwalls. The second note may be eliminated if the bars noted are fully detailed on the plans. In either case, the bars need to be included in the reinforcing steel schedule and estimated quantities.)* |||
9. The Contractor shall install Transition Barrier vertical closed stirrups as shown in Standard Details Section 526, prior to the placement of the curb concrete. |||
 10. Provide 3 additional stirrups in the curbs at each Transition Barrier location. |||

D.4 Standard Notes Piles

1. Piles marked with an arrow shall be battered XX in/ft in the direction of the arrow. ||
2. The maximum factored pile load is XX kips (including XX kips allowed for downdrag). ||
3. Estimate of piles required:

Abutment Number 1:	XX-HP XX x XX	@ XX ft
Abutment Number 2:	XX-HP XX x XX	@ XX ft
Pier Number 1:	XX-HP XX x XX	@ XX ft
Pier Number 2:	XX-HP XX x XX	@ XX ft

(The following note is used for integral abutments with steel stringers.) |

4. Piles shall not be out of position shown by more than 2 inches in any direction. |

(The following two notes are used for pile-supported foundations. The Geotechnical Designer will make a recommendation for their use or exclusion. The Structural Designer should determine the appropriate pay item and the Geotechnical Designer determine the number of dynamic tests.)

5. The Contractor shall perform and submit a wave equation analysis for review and acceptance by the Resident. The maximum allowable driving stress is 0.90 times F_y . The submittal analyses shall include the proposed stopping criteria based on the wave equation analysis and the proposed driving system. The stopping criteria shall include the blows per inch and the number of 1 inch driving intervals at which pile installation may be terminated. The cost of performing the wave equation analysis will be considered incidental to Item No. 501.92, Pile Driving Equipment Mobilization. ||
6. The Contractor shall perform XX dynamic load test(s) to confirm the ultimate capacity of the piles. The required nominal resistance for the pile is the factored axial pile load divided by a resistance factor of 0.65 per LRFD Specifications. The dynamic test shall be performed on the first production pile driven. ||
7. All piles shall be equipped with a pile tip in accordance with Standard Specification Section 501.10, Prefabricated Pile Tips. |
8. H-pile material shall be ASTM A572, Grade 50. |
9. Pipe pile material shall be ASTM A252, Grade 3, 45 ksi. |

D.5 Standard Notes Piers

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.
2. The maximum factored applied footing pressure is XX ksf.

Design Criteria

1. Critical AASHTO Load Combination – Strength, Service, Extreme Event, Fatigue Limit State.
2. Buoyancy - Water level assumed at Elevation XX.
3. Stream flow - Velocity of XX fps skewed at XX° to longitudinal centerline of pier.
4. Wind - XX mph or XX ksf.
5. Ice - Thickness X feet, pressure XX ksf at Elevation XX, 30% of nose force applied transverse to pier.

D.6 Standard Notes Seal Cofferdams

1. The seal concrete placement dimensions represent the minimum seal size necessary to meet design requirements and are not based on the use of any particular sheet pile section.
2. The horizontal pay limit for seal concrete shall be to the dimensions shown on the plans. No additional payment will be made for concrete placed outside of these limits.
3. When sheet piling is used for seal cofferdams, appropriate rolled corners shall be used, and the inside face of the sheet piling shall be at or outside of the seal concrete dimensions shown.
4. The depth of the seal is set for a water elevation of XX. If the water elevation at the time of construction is higher, the depth of the seal shall be adjusted.

(The following note is used seals without piles.)

5. The method of placing dowels in the seal concrete shall be approved by the Resident.

D.7 Standard Notes Structural Steel

1. Camber ordinates, as shown, are computed to compensate for all dead load deflections and for the curvature of the finished grade profile. ||

(The following two notes are used with welded girders only.) ||

2. No transverse butt-weld splices will be allowed in the flange plates or web plates within 10 feet or 10% of the span length (whichever is greater) from the points of maximum negative moment or maximum positive moment. Butt-weld splices in flanges shall be not less than 3 feet from transverse butt-welds in the web plates and no transverse web or flange butt-welds shall be located within 3 feet of other transverse welds (e.g. connection plates to web welds) on either flange or web. No transverse butt-weld splices will be allowed in areas of stress reversal.
3. Sections of flange plates or web plates between transverse shop splices or between a transverse shop splice and a field splice shall be not less than XX feet in length unless otherwise shown on the plans.

(The following note is used with haunched welded girders only.) ||

4. One longitudinal butt weld splice will be allowed in the web of the haunched sections of the girders. Feather edges between the longitudinal welds and the bottom flanges will not be allowed.
5. Bearing stiffeners shall be plumb after erection and dead loading of the structure. Intermediate web stiffeners may be either plumb or normal to the top flange.
6. Crossframe or diaphragm connection plates may be either plumb or normal to the top flange.

(The following note is used only with designs using A709, Grade 50 or painted Grade 50W.)

7. Filler plates may be steel conforming to the requirements of A709, Grade 36.

(The following note is used when more than one steel design is provided.) ||

8. The dimensions and elevations omitted from the Bottom of Slab Elevations table, the Camber Diagram, and the Stress Diagram will be provided to the Contractor for the structural steel option that has been selected.
9. At locations marked with an asterisk (*), the designated diaphragms shall be changed to a Type A (C) (D) diaphragm as required to accommodate the Contractor's deck placement sequence. No extra compensation will be allowed for any diaphragms so substituted, and any additional costs will be considered incidental to the Contract items. ||

10. Deleted.

||

11. Deleted.

||

(The following note is used when a single span rolled beam with 3" or more camber is used.)

12. The Contractor may substitute welded plate girders in place of the rolled beams shown on the plans, as approved by the Resident. The fabricator shall determine the plate thicknesses based upon the depth and moment of inertia of the rolled section.

D.8 Standard Notes Precast Concrete Superstructures

(The following note is used with 0.5 inch diameter strand.)

1. Prestressing strands shall be 0.5 inch diameter. The tensioning force is 31 kips per prestressing strand.

(The following note is used with 0.6 inch diameter strand.)

2. Prestressing strands shall be 0.6 inch diameter. The tensioning force is 44 kips per prestressing strand.
3. The top surface of the upper flange of the prestressed beams shall be raked to a surface roughness of +/- ¼ inch, except at locations corresponding to the blocking points. At these locations a flattened area of sufficient size shall be left to facilitate taking elevations for setting bottom of slab elevations.
4. The drilling of holes in the prestressed beams and the use of power-actuated tools on the beams will not be permitted.
5. Neoprene pads shall be either polychloroprene or natural polyisoprene of 50±5 Shore A durometer hardness, and shall conform to the requirements of Division 2, Section 18.2 of AASHTO Standard Specifications for Highway Bridges. Neoprene pads will not be paid for directly, but will be considered incidental to related Contract items.
6. Install a 1 inch diameter nonmetallic void drain in the bottom of each void at both ends.
7. Reinforcing steel shall have a minimum concrete cover of 2 inches unless otherwise noted.
8. Post-tensioning shall be covered by a seamless polypropylene sheath, with corrosion inhibiting grease between the strands and sheath, for the full length of the strand except at the anchorage location.
9. The Contractor shall calibrate the jacking equipment as necessary to provide an anchorage of 38 to 41 kips after setting losses in each 0.6" diameter post-tensioning strand.

(The following note is used for all voided slab and butted box beam structures.)

10. Screed rails shall be installed to the elevation shown on the profile, adjusted for wearing course thickness and cross slope.

D.9 Standard Notes Superstructures

1. Form a one inch V-groove on the fascias at the horizontal joint between the curb and slab.
2. Reinforcing steel shall have a minimum concrete cover of 2 inches unless otherwise noted.
3. Adjust reinforcing steel to fit around the bridge drains in a manner approved by the Resident. Do not cut transverse reinforcing bars.

(The following note is used for simple span structures.)

4. The superstructure slab concrete for each span shall be placed continuously and shall be kept plastic until the entire placement has been made.

(The following note is used for multiple span continuous structures with less than 250 yd³ of deck concrete.)

5. The superstructure slab concrete shall be placed in one continuous operation and the concrete shall be kept plastic one complete span behind the span being placed.

(The following note is used for multiple span continuous structures with more than 250 yd³ of deck concrete.)

6. Unless the superstructure slab concrete is placed in one continuous operation, the initial placement shall start at a simply supported end of the deck slab and shall terminate at the completion of a positive moment section. Successive placements shall proceed from the end of the previous placement, terminate at the completion of a positive moment section, and include two or more spans. Concrete in a placement shall be kept plastic one complete span behind the span being placed. A minimum of 5 days shall elapse between successive partial placements. The superstructure slab concrete placement sequence shall be approved by the Resident.

(The following note is used with staged construction of CIP structural slabs.)

7. The formwork and its supports, over the full width of the structural slab, shall remain in place until a minimum of 48 hours has elapsed after placement of the final section of the slab. After this period, removal of formwork for sections meeting the requirements for form removal of Standard Specifications Section 502, Structural Concrete, may proceed.

(The following note is used with granite curb on the superstructure.)

8. Mortar for bedding and for joints in the granite curb shall contain an approved non-shrink additive.

(The following two notes are used for seals where applicable. Seal types required should be noted on the plans. When compression seals are used, a Compression Seal Adjustment Chart should be shown on the plans)

9. The seals to be furnished shall have a minimum Movement Rating of:

Abutment Number 1 = XX

Abutment Number 2 = XX

10. The Resident shall approve the seals prior to fabrication of the Expansion Device.

(The following two notes are used when Precast Deck Panels are allowed.)

11. At the Contractor's option, Precast Deck Panels may be used in place of the full depth cast-in-place slab in accordance with Special Provision 502, Structural Concrete - Precast Deck Panels, and in accordance with the Standard Details.

12. Payment for the reinforcing steel fabricated, delivered, and placed in the cast -in-place portion of the structural concrete slab will be considered incidental to the appropriate Section 502 pay item.

13. The theoretical blocking used for design of the structure is XX inch(es) at the centerline of bearings of the abutments and piers. Refer to Standard Detail 502 (02) for blocking details.

14. The Contractor shall install Transition Barrier vertical closed stirrups, as shown in Standard Details Section 526, prior to the placement of the curb or sidewalk concrete.

(The following note may be eliminated if the bars noted are fully detailed on the plans. In either case, the bars need to be included in the reinforcing steel schedule and estimated quantities.)

15. Provide 3 additional stirrups in the curbs at each Transition Barrier location.

D.10 Standard Notes Elastomeric Bearings

1. The shear modulus of the elastomer shall be between 80 and 175 psi.
2. Vulcanizing of the elastomer to the steel plates shall be done during the primary mold process.
3. Upset the threads on the anchor rods after assembly.
4. Masonry plates, sole plates, and shear pins shall meet the requirements of ASTM A 709/A 709M, Grade 50 or 50W. Anchor rods shall meet the requirements or ASTM F 1554, Grade 105 and shall be swedged on the embedded portion of the rod.
5. Bearings shall be covered during transit.
6. Masonry plates shall be galvanized in accordance with Section 506. Sole plates for steel superstructures shall be treated in the same manner as the structural steel. Anchor rods, washers, nuts and shear pins shall be galvanized to ASTM A 153 or ASTM B 695, Class 50, Type 1.
7. The bearings are designed so that the superstructure may be erected when the ambient air temperature is within the range of 65°F and 90°F. If the ambient air temperature is outside this range, the bearings shall be reset as directed by the Resident.
8. Deleted.
9. All bearings shall be marked prior to shipping. The marks shall include the bearing location on the bridge, and a direction arrow that points up-station. All marks shall be permanent and shall be visible after the bearing is installed.

(The following note is used when bearings are to be welded to steel girders.)

10. All necessary precautions shall be taken to protect bearing components from field weld flash and spatter. Heat from welding operations shall be controlled such that steel adjacent to the elastomer does not exceed 200°F. The temperature shall be verified by the use of temperature indicating crayons or other suitable means.

D.11 Standard Notes HLMR Bearings

1. Refer to the Special Provisions for design, materials, fabrication, and general construction requirements.
2. The actual dimension "H" shall be the responsibility of the Contractor. Dimensions and sizes of plates not shown are dependent on design loads, bearing type, capacity, and the manufacturer of the bearings. The shop drawings, prepared by the manufacturer, shall provide all pertinent bearing information. The final bridge seat elevations shall be determined by the Contractor and submitted with the shop drawings for approval prior to construction of the substructure units.
3. Masonry plates shall be placed on 1/4" thick preformed pads in accordance with the specifications.
4. All steel, except anchor rods, shall be AASHTO M 270, Grade 70W.

(Edit "Grade 55" if a higher strength anchor rod is used.)

5. Anchor rods shall meet the requirements or ASTM F 1554, Grade 55, and shall be swedged on the embedded portion of the rod.
6. Anchor rod spacing shall be coordinated with the bearing manufacturer.
7. Bearing installation shall be in strict conformance with the Special Provisions and the manufacturer's recommendations.
8. The abbreviation "PTFE" indicates polytetrafluoroethylene.
9. The design temperature range shall be 150°F (-30°F to 120°F)
10. At abutment bearings only, all steel located below the PTFE sliding surface shall be coated in accordance with Special Provision, Section 506, Protective Coating-Steel (Thermal Spray Coating). All remaining steel at abutment bearings shall be coated in accordance with Special Provision, Section 506, Protective Coating-Steel (Zinc Rich System).
11. All bearings shall be marked prior to shipping. The marks shall include the bearing location on the bridge, and a direction arrow that points up-station. All marks shall be permanent and shall be visible after the bearing is installed.

(The following note is used if applicable.)

12. Bearings need not be designed with hold-downs.

D.12 Standard Notes Structural Plate Structures

1. One X'-X" diameter Structural Plate Pipe is required. Top plates shall be 0.XXX inch thick, and bottom (three) plates(s) shall be 0.XXX inch thick. The pipe shall be elongated 5% vertically.
2. One XX'-X" span by X'-X" rise Structural Plate Pipe Arch required. Top plates shall be 0.XXX inch thick, and bottom and corner plates shall be 0.XXX inch thick.
3. Ends shall be cut on a 1:1.75 bevel normal to the end skew shown on the details.
4. Riprap adjacent to the pipe shall be carefully placed so as not to damage the pipe and so that the finished slope will match the ends of the pipe. Any extra labor, material, or equipment used will be considered incidental to Item 610.08, Plain Riprap. Any damage done to the structure during construction shall be repaired or replaced as determined by the Resident at the Contractor's expense.
5. Place a 24 inch wide strip of Temporary Erosion Control Blanket along the top of the riprap and over the structure, typical at both ends.

(The following note is used with an aluminum pipe or pipe arch.)

6. End reinforcement devices shall be of aluminum and shall be of sufficient strength to provide a minimum section modulus, about an axis perpendicular to the center of the pipe of $1.10 \text{ in}^3/\text{ft}$ of pipe circumference. Maximum spacing of the devices shall be 5'-5". Attachment to the pipe shall be with 3/4" galvanized steel bolts. Section properties and details of the device and the method of attachment shall be submitted to the Resident for approval. Payment for end reinforcement devices will be considered incidental to the structural plate structure pay item.
7. The structural plate structure shall be constructed in the dry. The approximate weight of the structure is XXXX pounds.
8. Granular Borrow shall meet the requirements of Subsection 703.19, Material for Underwater Backfill.

D.13 Standard Notes Drilled & Anchored Bolts and Reinforcing Steel

(The following note is used for Type 1 anchors when bolts are size 7/8" or greater.)

1. For drilling and anchoring bolts size 7/8" or greater, the anchor material chosen from the prequalified list shall be submitted to the Resident for approval.

(The following note is used for Type 3 anchors when reinforcing bars are size #9 or greater.)

2. For drilling and anchoring reinforcing bars size #9 or greater, the anchor material chosen from the prequalified list shall be submitted to the Resident for approval.

D.14 Standard Notes Precast Concrete Arches or Boxes

(The following note is used if applicable.)

1. The precast units shall be designed to carry construction loadings with a minimum fill cover of 18 inches on top of the units. |
2. The construction, handling, and assembly of the precast units shall be in accordance with Special Provision Section 534 Precast Structural Concrete, and with the Manufacturer's Specifications as applicable.
3. Install standard membrane waterproofing over the top and to 12 inches down the exterior sides of the precast units. |

D.15 Standard Notes Prefabricated Concrete Modular Gravity Wall

1. The Contractor shall provide a Prefabricated Concrete Modular Gravity (PCMG) wall in accordance with Special Provision 635. The PCMG shall be designed and stamped by a Registered Professional Engineer and the design shall be submitted to the Resident for review. Plan Details are shown for estimating purposes only.
2. The precast units shall be manufactured by the following, or equal:

"T-Wall" as manufactured by a licensed manufacturer of Neel Company.

"DoubleWal" as manufactured by a licensed manufacturer of DoubleWal Corp., Plainville, Connecticut.
3. The factored bearing pressure for PCMG walls shall not exceed the factored bearing resistance of XX ksf for the strength limit state. The factored bearing pressure for the service limit state shall not exceed the factored bearing resistance of XX ksf.

(The following note is used when the bridge passes over salt water. Edit "corrosion resistant steel" to specify type of bar as described in Section 6.2.1.2 Reinforcing Steel.)

4. The PCMG wall shall consist of Class "LP" concrete and corrosion resistant reinforcing steel.

(The following note is used when cofferdams are required.)

5. Cofferdams for the PCMG wall installation shall be included with Pay Item 511.07, Cofferdam.

D.16 Standard Notes CIP Box Culverts

1. Form a 1 inch V-groove at the front face of vertical contraction and construction joints.
2. Reinforcing steel shall have a minimum concrete cover of 2 inches unless otherwise noted. |
3. Place 4 inch diameter drains in the walls and wingwalls at 10 feet maximum spacing. The exact location will be determined by the Resident. |
4. Granular Borrow shall meet the requirements of Subsection 703.19, Material for Underwater Backfill.
5. Granular Borrow under the bottom slab may be reduced or omitted if the Resident determines that the existing material is suitable.
6. Cover the vertical contraction and construction joints on the back side in accordance with Standard Detail 502(01). Cover the contraction joints in the top slab in the same manner, but without recessing the concrete. |