Alaska Department of Transportation and Public Facilities

Alaska Bridges and Structures Manual

March 1, 2023

Alaska Bridges & Structures Manual

March 2023

FOREWORD

The *Alaska Bridges and Structures Manual* presents the Department of Transportation & Public Facilities' (DOT&PF's) typical structural design policies and practices. All bridge engineers should meet the criteria presented in the *Manual* and must request exceptions to the *Manual* criteria when conditions warrant. Engineers should consider economic impacts, aesthetics, and the social and cultural resources of the project area and fulfill DOT&PF's mission of providing a safe and efficient transportation system. Because it is impossible to address every issue that bridge engineers will encounter, exercise sound engineering judgment when conditions arise that are not specifically covered in the *Manual*.

The term "bridge engineer" and "bridge designer" are used interchangeably throughout the *Manual*. If needed, any distinction between these two terms should be based on context.

The *Alaska Bridges and Structures Manual* was initially prepared based on the 6th Edition of the AASHTO *LRFD Bridge Design Specifications* and has been updated to conform with the 9th Edition.

REVISION PROCESS

The *Alaska Bridges and Structures Manual* is intended to provide current structural design policies and practices for use in developing DOT&PF projects. Revisions to this Manual will be released on an annual basis as needed and after approval by FHWA. The format of the revisions will be in the form of replacement or insert pages to the existing Manual. The updated pages will include the date of the release in the bottom footer of the page. The revisions will be handled as interims and the Manual will be republished as deemed necessary.

It is the responsibility of the *Manual* holder to keep the *Manual* updated.

The DOT&PF Bridge Section will evaluate changes in the structural design literature (e.g., updates to the *LRFD Specifications*, the issuance of new research publications, revisions to federal regulations) and will ensure that those changes are appropriately addressed through the issuance of revisions to the *Manual*. Bridge engineers have a responsibility to remain current with the AASHTO *LRFD Bridge Design Specifications* revisions until the *Manual* is updated. It is important that users of the *Manual* inform DOT&PF of any inconsistencies, errors, need for clarification, or new ideas to support the goal of providing the best and most up-to-date information practical. Comments and proposed revisions may be forwarded to the Chief Bridge Engineer using the Revision Proposal Form.

Alaska Bridges and Structures Manual

Revision Proposal Form

To propose a revision to the *Alaska Bridges and Structures Manual*, complete and return this Revision Proposal Form to:

Chief Bridge Engineer
Bridge Section
Alaska DOT & Public Facilities
P. O. Box 112500
Juneau, Alaska 99811-2500

Identification

Date Submitted: ____

Submitted By (name, agency/firm): ______________________________

Contact Information (phone #, e-mail): ___________________________

Description of Proposed Revision (attach additional sheets as necessary)

Applicable *Manual* Section Number(s): ___________________________

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Proposed Revision: ____

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Justification for Revision: _______________________________________

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Table of Contents

Part I – Administration and Procedures

Part II – Structural Design

Part III – Existing Bridges/Bridge Operations

1. DOT&PF Organization (Reserved)

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2. Bridge Project Development Process

The bridge project development process is detailed in Chapter 4 of the *Alaska Highway Preconstruction Manual* as part of the larger highway project development process.

This chapter of the *Alaska Bridges and Structures Manual* is reserved for any future information not covered by the *Alaska Highway Preconstruction Manual*.

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3. Administrative Policies and Procedures (Reserved)

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4. Bridge Type Selection Report

Bridge type significantly influences structure performance, functionality, and long-term maintenance. The bridge type selection process involves evaluating many features to identify the most appropriate bridge type for the site.

Potentially relevant features include the elements of the bridge (e.g., foundations, abutments, piers, girders, bearings, expansion joints), materials (e.g., concrete, steel), and geometrics (e.g., clearances, structure depth, structure width, span lengths). High-cost features or those with a “fatal flaw” should be eliminated early in the evaluation process.

The Bridge Type Selection Report documents the findings of this selection process and recommends the most appropriate bridge type.

4.1. Bridge Type

4.1.1. Simple Bridges

Bridge type selection for simple bridge projects is straightforward, and the preparation of a formal Bridge Type Selection Report is not necessary. In these cases, the bridge engineer or consultant will send this determination to the Chief Bridge Engineer for approval. Then, the Chief Bridge Engineer submits a project package to the regional project manager. This submission will include the following:

- a cover memorandum (see Figure 4-1 for a sample)
- a description of the bridge
- a cost estimate based on preliminary quantity calculations, and
- General Layout and Site Plan sheets for the bridge.

4.1.2. Complex Bridges

Due to size, location, terrain, environmental considerations, local preferences, or other factors, the ideal bridge type may not be apparent. For these more complex bridge projects, prepare a Bridge Type Selection Report to identify feasible structure alternatives.

In this report, evaluate each viable bridge type considering initial and long-term costs, constructability, serviceability, and compatibility with the site. Prepare a cost estimate for each alternative. The Report typically includes a recommendation for the preferred bridge type.

MEMORANDUM

State of Alaska

Department of Transportation & Public Facilities
Statewide Design & Engineering Services Division/Bridge Design

TO:	Sarah Schacher, P.E. Project Manager Northern Region	**DATE:**	January 13, 2011
		BRIDGE NUMBER	1520
		TELEPHONE:	465-2975
		FAX:	465-6947
FROM:	Richard A. Pratt, P.E. Chief Bridge Engineer	**TEXT TELEPHONE:**	465-3652
CONTACT:	Elmer Marx, P.E. (907) 465- 6941	**SUBJECT:**	Preliminary Design Holden Creek Bridge

As requested, we have prepared preliminary General Layout and Site Plan drawings for the subject crossing.

The preliminary cost estimate for the proposed bridge is attached. The cost estimate includes all materials and labor for the bridge related pay items as well as 10% for mobilization and demobilization, 15% for construction engineering, and 4.79% for ICAP. The unit bid prices are based upon the recently bid Oksrukuyik Bridge that is very similar to the proposed bridge and relatively close in location. Based upon the recent bid tab data, the estimated cost of the Holden Creek Bridge is quite a bit greater than is normally anticipated.

We do not yet have the foundation or hydraulic recommendations for this site. Also, the roadway plan, profile and typical section data have not been finalized. Consequently, a 25% contingency is included. As information becomes available, we will incrementally decrease the contingency value (percentage) until we provide the final bridge cost estimate.

The proposed roadway geometry requires replacement of the existing bridge on the same alignment. A detour structure will be required to accommodate traffic during construction of the replacement bridge. Although we suspect that a culvert(s), ice road, or other non-bridge option may be feasible at this location, we have included the cost of a detour bridge in the preliminary estimate.

The existing structure has a history of hydraulic related problems. Maintenance personnel reported that approximately 15 years ago the bridge was pushed downstream during a high flow event. Also, the bridge is considered scour critical (NBI Item 113). In order to address these problems, we propose to raise the roadway profile grade by about eight feet. The resulting bridge provides significantly more vertical and horizontal clearance and should eliminate future problems.

Please contact Elmer if you have any questions.

EEM/bm

Figure 4-1
Cover Memorandum

4.2. Procedures

4.2.1. Use of Manual Chapters

Several other chapters within the *Alaska Bridges and Structures Manual (Manual)* are important for identifying, evaluating, and selecting the bridge type. Chapter 11 “Structural Systems and Dimensions” is especially useful in the evaluation process.

The remaining chapters in Part II “Structural Design” of this *Manual* are predominantly directed toward the detailed design of the structural elements; however, these chapters may present DOT&PF policies and practices that impact bridge type selection.

4.2.2. Distribution/Approval

The bridge engineer or consultant prepares the Bridge Type Selection Report and coordinates with applicable DOT&PF sections when conducting the study (e.g., Roadway Design, Hydraulics). The report includes a cover memorandum that is submitted to the Chief Bridge Engineer for approval. The Chief Bridge Engineer signs and forwards the memorandum and report to the regional project manager.

4.2.3. Design Study Report

Section 450.5.2 of the *Alaska Preconstruction Manual* discusses the Design Study Report (DSR), which documents the basis for the preferred design alternative. The regional project manager prepares the DSR. Typically, the Bridge Type Selection Report will be an Appendix in the DSR.

4.2.4. Study Initiation

A “Start-Up Package” from the regional project manager or highway designer is required to begin the preliminary bridge layout, type selection, and cost estimates needed for the type selection process. The “Start-Up Package” includes:

- proposed roadway plan & profile data
- topographic data in the vicinity of the bridge site including a hydraulics survey
- roadway typical section
- right-of-way limits
- utility locations and utilities to be carried on the new bridge
- environmental design criteria and commitments; and
- preliminary hydraulic and geotechnical recommendations, when available

4.3. Report Format and Content

In general, prepare the Bridge Type Selection Report in the sequence and format discussed below. All topics may not be required for every structure site. Provide sufficient detail for the reader to understand the decision-making process.

Cover Page

Include the bridge name, location, and bridge number. Include the title of the report (i.e., Bridge Type Selection Report) directly beneath the identification information. On the bottom of the page, include the author's contact information, date of the report and engineering seal (if required).

Executive Summary

On the first page of the report, provide a one or two paragraph summary that identifies the purpose of the bridge type study, location of the bridge, number of alternatives studied, and types of structures considered. Identify the preferred alternative and list the evaluation criteria used to select this structure type.

Table of Contents

Provide a table of contents for the major sections and appendices of the report.

Introduction

Briefly describe the history and purpose for the project. If the project involves a bridge replacement, describe the existing structure and why the existing bridge needs to be replaced. Indicate the overall width and length, span lengths, skew angle, superstructure, substructure, and foundation types. Also, identify the existing vertical clearance if the bridge is located over a road or railroad.

Site Conditions

Discuss the following, as applicable:

1. Location. Identify the general location of the bridge (e.g., distance and direction from towns, lakes, major rivers or landmarks; milepost number on existing highways). Use GoogleEarth, a USGS map, etc., to identify the location if appropriate.

   Also, include a general description of the terrain and any special structural features required due to the terrain (e.g., deep cuts, long spans, high fills). If the terrain varies significantly among alternatives, describe the terrain for each alternative.

2. Bridge and Roadway Alignment. Describe the new structure's alignment in relation to the existing alignment, if applicable. State if the horizontal alignment will be located on a curve or tangent. Describe the alignment (e.g., grades, vertical curve, superelevation transition). If the proposed alignment varies among alternatives, describe the alignment for each alternative. See the *Alaska Preconstruction Manual* for DOT&PF policies and practices on road design.

3. Size. Document the proposed width and length of the bridge. See the *Alaska Preconstruction Manual* for DOT&PF criteria for bridge widths. Structure length calculations are found in Section 11.8 of this *Manual*.

4. Vertical Clearances. Discuss the minimum vertical clearances over roads or the water elevation based on the design flood event. See the *Alaska Preconstruction Manual* for vertical clearance requirements. If the structure is over navigable water, discuss the navigational clearances (horizontal and vertical) required by the US Coast Guard.

5. Foundation/Soil Conditions. Describe the soil conditions at the structure site and how they affect the type selection and design of the foundation.

6. Hydraulic Conditions. Section 11.2 discusses the objectives and nature of the Hydraulics Report prepared by the hydraulics engineer. Use the preliminary hydraulic analysis to indicate the required channel dimensions and bridge waterway opening. Address existing and anticipated scour issues.

7. Seismic Acceleration. Include the seismic parameters for the site as described in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and state if soil liquefaction may be an issue. If available, discuss how past seismic events at the site have damaged existing structures. See Section 11.4.6.

8. Material & Equipment Transport. Explain how the construction materials and equipment can be transported to the site for at least one alternative. Describe how transport issues affect the design and type selection (e.g., weight, length).

9. Miscellaneous. Describe other site conditions that may affect the design and selection of the structure, including:

   a. utilities
   b. right-of-way
   c. fish habitat
   d. environmental issues
   e. aesthetics
   f. construction
   g. potential for future widening

Bridge Type Summary

For each alternative considered, document the following information:

- foundation, substructure and superstructure type
- structure dimensions (e.g., overall length, skew, deck geometry, number of spans, span lengths)
- abutment and pier location and size
- anticipated seismic performance
- constructability (e.g., weight, special equipment requirements, historic experience)
- construction phasing
- maintenance considerations
- special requirements (e.g., utilities, temporary falsework, temporary bridges)
- transport issues
- aesthetics
- right-of-way
- initial and long-term costs
- recommendation for bridge type selection

Summary of Results

Provide a summary of the alternatives considered. Present the alternatives in a tabular format. For each alternative include the title, a very brief description, estimated cost, and advantages and disadvantages.

Preferred Alternative

Identify the preferred alternative. Summarize the positive and negative features of the recommended configuration and the reasons for its selection. Include preliminary General Layout and Site Plan sheets in the Bridge Type Selection Report. Show any existing structures with foundation elements and their relationship to the new bridge.

The Chief Bridge Engineer recommends the final bridge type to the regional project manager.

Appendices

Use of appendices is acceptable to reduce the size of the report or to provide additional information. Some topics that may be addressed in the appendices include:

- decommissioning existing bridge
- proposed construction procedures
- itemized bridge cost estimates
- life-cycle cost calculations
- illustrations of the substructures and superstructures considered
- construction sequences

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5. Drafting Guidelines

Bridge plans sheets should be similar in appearance, content, and style. Follow the standards in this chapter for any plan set containing a bridge design.

Submit completed bridge designs to the Bridge Section in AutoCAD format in addition to generating hard copy plan sheets for the contract.

5.1. CADD File Requirements

Prepare and submit all drawing files in latest release of AutoCAD. Use the bridge number as the file name. For revised plans, use the bridge number followed by an "R" in the file name. All reference files that are part of the finished plan sheet must be merged into a master file. Do not use reference (xREF) files; therefore, detach all reference files after merging needed files and details. Purge all elements that are not part of the final plan sheet and remove all elements that do not reside within the boundaries of the sheet border.

5.1.1. Design File

The global origin, in all Bridge Section AutoCAD drawings, is defined as X,Y=0,0 in the lower left corner of the design plane.

US Working Units

The AutoCAD format for US working units is the following:

- Set Master Units to decimal feet (ft).
- Make all drawings to scale in model space at 1=1.

Metric Working Units

The AutoCAD format for metric working units is the following:

- Set Master Units to millimeters (mm).
- Label elevations and stationing in meters.
- Make all drawings to scale in model space at 1=1.

5.1.2. Layer Assignment

Use layer assignments shown in Table 5-1. If additional layers are needed, use a layer name that clearly conveys the content of the layer; do not create obscure abbreviations.

Table 5-1
Layer Assignments

Layer Name	Color	Layer Content
PEN1	White	Existing Structure Lines, Utilities
PEN2	Yellow	North Arrow, Reinforcement, Note Text
PEN3	Green	Main Object Lines, Title Text
PEN4	Blue	Items to stand out, Sheet Border
PEN5	Cyan	Dimension Lines, Centerlines
PEN6	Magenta	Hatching, Existing Reinforcing Steel
V_PORT	Red	View Ports, All Items not being printed

5.1.3. Linetypes

Use the linetypes shown in Table 5-2 to distinguish the type of object being drafted. Use a global line scale (AutoCAD command "ltscale") of 0.5. Individual settings of linetype scale are acceptable if needed.

5.1.4. Text

Present all text as left justified, except titles are center justified. When one line is not sufficient for text, use multiline text (AutoCAD command "mtext") to ensure the proper spacing between lines.

Normal Text

Use the following AutoCAD setting for normal text in the drawings:

- Font = Simp1 (obliquing = 22)
- Text Height = 0.13 x Scale of Detail

Table 5-2
Standard Linetypes

Functional Application	Layer	Linetype
Centerlines	PEN5	Center, Center2
Existing Structures	PEN1	Hidden, Hidden2
Drawings – Object lines	PEN3	Continuous
Drawings – Reinforcement	PEN2	Continuous
Ordinary High Water	PEN1	Phantom2
Utilities	PEN1	*Specific to Type of Utility*
Hidden Lines	PEN1	Hidden, Hidden2
Dimensioning	PEN5	Continuous
Text – Normal	PEN2	Continuous
Text – Headings, etc.	PEN3	Continuous

Text for Headings, Titles, Views, etc.

To format view titles use all capital letters, center-justified and underlined with two lines. Extend the lower line one character past the title on each end; make the upper line the same length as the title. Place the bar scale under the title without crowding. Use the following AutoCAD settings for text in titles and headings:

- Font = Leroy (obliquing = 0)
- Text Height = 0.1563 (*Paper Space* only)

5.1.5. Dimensioning

Use PEN5 (cyan) on all dimension lines, leader lines, and centerlines. Use curved leader lines for arrowed notes. Keep the use of “S” shaped leader lines to a minimum. Do not put text or dimensions in the body of detail. For stacked dimensions between extension lines, separate the dimension lines a distance of two times the text height. For stacked dimensions between extension lines with text placed both above and below a dimension line, separate the dimension lines a distance of three times the text height. Provide a consistent distance between dimension lines throughout the plan.

Metric Dimensioning

Use millimeters only for all dimensions on metric plan sets. Use meters for elevations and stations.

5.1.6. Hatching

Use layer PEN6, magenta for hatching. Use hatching patterns for riprap, earth, existing structures, and other items, as needed. Do not hatch concrete.

5.1.7. Title Blocks and Borders

Use the Bridge Section standard title block for all bridge and structures drawings. See Chapter 6 for more information about title blocks, borders, and sheet organization.

All sheets will show the name of the person responsible for designing, drafting, and checking that sheet. The Professional Engineer Seal provided by the Alaska State Board of Registration for Architects, Engineers and Land Surveyors is placed in the title block of the drawings only when the entire set of drawings is complete and the bridge engineer is ready to sign the drawings.

5.1.8. Plotting

Use the files “Bridgehalf.ctb” and “Bridgefull.ctb” for plotting. By using these files, the proper plotter setting are automatically configured. The “Bridgehalf.ctb” settings are used for half-size plots (e.g. 11” x 17” size). The “Bridgefull.ctb” settings are used for full-size plots (e.g. 22” x 34”). The newest version of these files is available upon request from the Bridge Section.

Final sheets should be readable when plotted at both full-size and half-size. Review the plotted sheet in addition to the screen views to verify quality before submitting a set of drawings.

5.2. Drafting Standards

Effective drafting requires care and attention to detail. In addition to the requirements of this section, use the following general guidelines to enhance the effectiveness of drafted material:

1. Do not crowd sheets with so many drawings that clarity is compromised. Provide enough empty space around details to distinguish between adjacent details. Use additional sheets instead of crowding too many details onto one sheet.
2. Balance the empty space of plan sheets to avoid having one crowded and another mostly empty.
3. Label the Working Line (e.g. ℄ Brg., ℄ Roadway, ℄ Layout Line) consistently throughout the plans.
4. Tie dimensions to Working Points.
5. Keep abutments, piers, superstructure, etc., details together.

5.2.1. Scales

Draw all details to scale. Use a standard scale and the maximum scale practical. Use bar scales as much as practical. Place the bar scale under the title of the detail, both of which are centered below the detail.

Use only the following scales in drawings:

1. US Scales

Architectural: 3”=1’-0”, 1 1/2”=1’-0”, 1”=1’-0”, 1/8”=1’-0”, 1/4”=1’-0”, 3/16”=1’-0”, 1/2”=1’-0”, 1/16”=1’-0”

Engineers: 1”=10’-0”, 1”=20’-0”, 1”=30’-0”, 1”=40’-0”, 1”=50’-0”

2. Metric Scales

1=10, 1=20, 1=25, 1=40, 1=50, 1=75, 1=100, 1=125, 1=150, 1=200, 1=250, 1=300

5.2.2. Details and Notes

When drafting details and notes adhere to the following:

1. Do not mix a detail between *Paper Space* and *Model Space*.
2. Place a period at end of each sentence in the general notes. For arrowed notes use a period only if the note contains more than one sentence.
3. In general notes capitalize only the titles of books and manuals. For arrowed notes capitalize the first word of the note and nouns referring to bridge components.
4. Use “No Scale” as shown. Do not use “Not to Scale”, “NTS”, or other variations.
5. Use “Dwg. No.” instead of “Sheet No.” Do not use the number symbol “#” unless there is not enough room to write “No.”
6. Do not use spaces when using symbols (i.e. 2’-0”±, 2’-0”ø). When labeling steel shapes do not use spaces or dimension symbols except for the length (i.e. W24x76, L6x6x3/4x2’-6”).
7. If a plan sheet contains numerous variable dimensions and other data, especially for framing plans and beams, use tables to keep data in order.
8. Use active voice in all notes.
9. Verify that the details are applicable and are to scale when copying details from other plans.
10. Use large-scale corner details on skewed bridges.
11. Leave extra lines in the “Summary of Quantities” and “Reinforcing Steel Schedule” tables for additions.
12. Label girder center lines “Girder A, Girder B, etc.” on the Framing Plan and other drawings. Label the girders from left to right looking ahead on station.
13. Cross-reference plan sheets by sheet title, not by sheet number.
14. Use extra details for uncommon work, but avoid unnecessary repetition of dimensions, details and/or notes. Do not double dimension.
15. When showing an existing bridge or structure on a plan sheet provide a hidden linetype to outline the relevant position.

16. Use isometric views sparingly and only when necessary for clarity.
17. Number all detail notes. Place all notes together on the bottom right hand side of the sheet. Do not enlarge details, such as bar bends, just to fill up space.

5.2.3. Symbols

Some commonly used symbols and the equivalent AutoCAD “Simpl” command code are shown in Table 5-3. Symbols can only be created in text mode; they must be copied into “mtext”.

Table 5-3
Common AutoCAD Symbols

Symbol	Code
+/-	%%P
℄	%%130
⅊	%%133
ø	%%C
° (degrees)	%%D
_ (underline)	%%U (at the start and end of word)
1/16	%%201
1/8	%%202
3/16	%%203
1/4	%%204
Additional fractions in increments of 1/16	%%204 +1 for each subsequent increment of 1/16

5.2.4. Abbreviations

Use abbreviations consistent with those from previous plans. Show a list of abbreviations applicable to the project on the SITE PLAN sheet. Table 5-4 gives a sample list of abbreviations.

5.2.5. Retrofit, Rehabilitation, and Staged Construction

On projects with staged construction, clearly indicate how the bridge construction is to be coordinated with staging, especially for complex projects. When drafting removal items, clearly indicate removal limits. Use a straight line to show a saw cut.

On retrofit and rehabilitation projects, denote elevations and dimensions taken from the as-built plans with a ± symbol. The ± symbol indicates approximate dimension. Require the contractor to verify these dimensions, elevations, etc. in the field if necessary, before using them on new work. Include this note under the General Notes of all rehabilitation projects:

“Existing stations, elevations and dimensions are based on as-built plans and those plans may not show existing dimensions and conditions. Where dimensions of the proposed work depend on the existing bridge dimensions, field-verify the controlling dimension and adjust proposed dimensions of the work to fit existing conditions.”

On subsequent sheets of all rehabilitation projects use the following note:

“Verify controlling field dimensions before ordering or fabricating any material.”

5.2.6. Detailing Reinforcing Steel for Concrete

The following standards apply to drafting reinforcing steel in concrete:

1. Use layer PEN2 (yellow) to draw reinforcing steel.
2. Detail reinforcement with one line not two lines.
3. For crowded views, do not show every reinforcing steel bar. Point to the first and last bar.

Table 5.4
List of Common Abbreviations

℄	= Centerline	F	= fixed bearing
℘	= Plate	f.f.	= far face
&	= and	Hwy.	= highway
@	= at	Jt.	= joint
ø	= diameter	ksf	= kips per square foot
±	= plus or minus	Lt.	= left
A/C	= asphalt concrete	max.	= maximum
Approx.	= approximate	min.	= minimum
Abut.	= Abutment	n.a.	= not applicable
Bot.	= bottom	n.f.	= near face
Br.	= bridge	No.	= number
Btwn.	= between	o.c.	= on center
Brg.	= bearing	OHW	= ordinary high water
C.I.P.	= cast in place	psi	= pounds per square inch
CJP	= complete joint penetration	ROW	= right of way
Clr.	= clear, clearance	Rt.	= right
Dia.	= diameter	S.I.P.	= stay in place
DHW	= design high water	spc.	= space, spaces
Dwg.	= drawing	Sta.	= station
Elev.	= elevation	Symm.	= symmetric
e.f.	= each face	Typ.	= typical
E	= expansion bearing	UT	= ultrasonic testing

4. When showing reinforcing steel sections, use an oversized circle (AutoCAD command "donut" with inside diameter = 0).
5. On a skewed bridge with skew angle greater than 20 degrees, detail main reinforcing steel spacing in the deck slab along edges of the slab. Do not detail spacing normal to the reinforcing steel, as this tends to increase the actual spacing beyond design spacing. Show and dimension a radius only on nonstandard bends.
6. Do not show reinforcement in details with scales less than 3/8"=1'-0".
7. Use the reinforcing steel identification convention shown in Figure 5-1. Start the numerical sequence at "01" with no gaps in the numerical order. For example, the first #4 reinforcing bar for an abutment would be labeled "A401".

Provide a reinforcing schedule (bar list) on all sheets containing reinforcing steel. However, if two or more sheets are needed for one bridge component (e.g. PIER and PIER DETAILS) show the reinforcing schedule on the first sheet only. Use actual reinforcing steel shapes in the reinforcing steel schedule.

5.2.7. Drafting Quality Control

Drawings should be thoroughly reviewed by the drafter as well as the bridge engineer at each stage of development.

Verify that details, data, and other information, which may be given on more than one plan sheet, agree on all locations of the plan sheet(s). Avoid double dimensioning as much as practical.

Carefully check plans for errors and spelling. Errors on Preliminary Plans could possibly be carried through the entire final plan stage without being discovered, and major last-minute revisions might then be necessary, resulting in disrupted schedules.

Verify that pay item numbers and descriptions in the Estimate of Quantities Table have the exact wording and spelling as the pay items in the DOT&PF *Standard Specifications for Highway Construction* and relevant standard modifications and special provisions.

Read a plan from a contractor's perspective to determine if the plan contains all the necessary information and data needed to construct the bridge. Verifying material quantities is often a good way to check for completeness of a plan sheet. If the quantities cannot be calculated then more information is required on the plans.

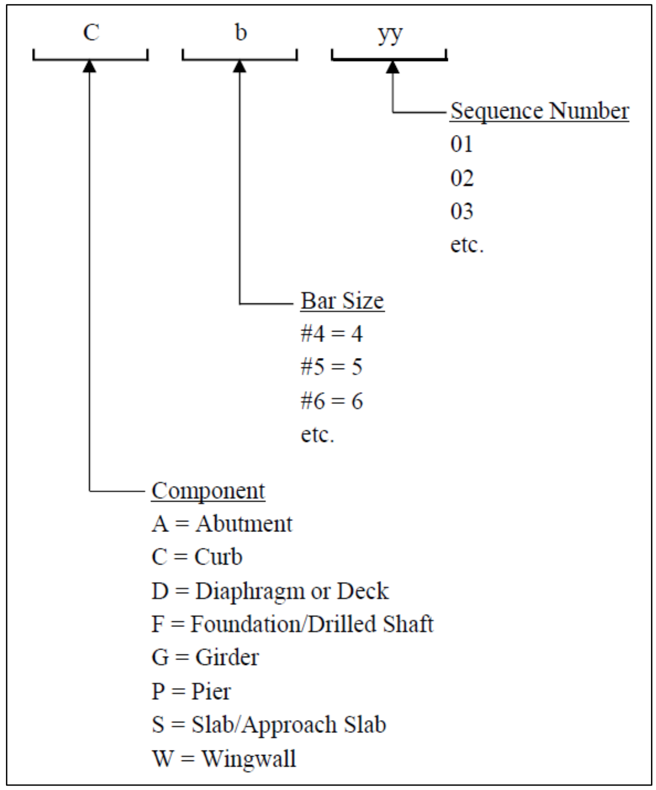

Figure 5-1
Reinforcing Bar Identification

5.3. Plan Sheets

Table 5-5 shows the order of plan sheets for a typical new structure. Variations in plan sheet titles and order can occur depending on project size and complexity. In general, plan sheets are placed in the order a bridge is constructed, from the foundation up. Plan sheets can be added or removed to accommodate the particular bridge.

5.3.1. Final Plan Sets

Plot final plan sets on mylar and have the mylars signed by the appropriate bridge engineer and checker(s). Create a digital copy in PDF format of the signed final mylar set. Increasingly, the regions are requesting that final plans be submitted in electronic format. If the mylars are not submitted to the region, store them in the vault flat file archives. Consultants must submit signed mylars to the Bridge Section along with the AutoCAD files and electronic PDF files.

Table 5-5
Typical Plan Sheets

Sheet Title	Sheet Contents
TITLE SHEET	Use when a bridge is let separately from roadway work, or when work is being completed at different locations.
GENERAL LAYOUT	General Plan, Elevation and Typical Section, Estimate of Quantities Table, Index to Plans.
SITE PLAN	Site Plan, General Notes, Pile or Footing Data Table, Estimate of Quantities Table (if does not fit on GENERAL LAYOUT).
RIPRAP LAYOUT	Riprap and Hydraulics information if not located on the SITE PLAN.
FOUNDATION PLAN	Footing and pile layout. Use this sheet when the SITE PLAN is too crowded.
MSE WALLS	MSE Wall layout and details. Use only with jobs that have MSE Walls.
ABUTMENTS	Abutment geometries, piles, and dowels to Working Points. Use an additional abutment sheet if abutments are not similar. Title ABUTMENT 1 and ABUTMENT X.
ABUTMENT DETAILS	Details pertaining to abutments.
WINGWALLS	Wingwall layout and details.
PIERS	Pier geometries, piles, and dowels to Working Points. Use additional pier sheets if piers are not similar. Label PIER 2 and PIER 3, etc.
PIER DETAILS	Details pertaining to piers.
TYPICAL SECTION	Typical section details, including diaphragm details.
FRAMING PLAN	Girder layout, Diaphragm spacing, Rail Post spacing, and Shear Connectors are to be labeled. May be combined with the TYPICAL SECTION Sheet.
GIRDERS	Girder geometry and reinforcing.
GIRDER DETAILS	Additional details that do not fit on girder sheet.
DECK	Deck details and reinforcing.
APPROACH SLAB	Approach slab details and reinforcing.
STEEL BRIDGE RAILING	Railing details. The sheet name should be consistent with the Item 507 description in the *Alaska Standard Specification for Highway Construction*.
UTILITIES	Utility details attached to bridge.
TEST BORING LOG AND LOCATIONS	Sheet provided by the foundation engineer with boring log information.

6. Plan Preparation

This chapter standardizes the content and appearance of bridge plans. The objective is to produce consistent drafting and bridge plans, because consistency enhances constructability of bridge projects. Exceptions to the standard appearance of the drafting and plan layout should only be used in unique circumstances.

The following sections provide a general description of the content and organization of each type of plan sheet. An example plan sheet is included for each section to demonstrate the layout and content. Appendix A has checklists to assist in verifying plan sheet completeness.

6.1. General Layout Sheet

6.1.1. Plan

Place the plan view in the lower half of sheet toward the left-hand side.

The preferred scale is 1"=20'-0", but the 1"=30'-0" scale may be used for larger structures. For smaller structures use 1"=10'-0". Avoid 1"=40'-0" or 1"=50'-0" as these scales become crowded on reduced plans.

For large structures, draw a detail on the GENERAL LAYOUT sheet at a small enough scale, such as 1"=200'-0", so the entire structure fits, and create a STRUCTURE PLAN sheet with the structure drawn at a 1"=20'-0" scale.

A station line or reference line tied to each structure on the sheet is required. The acceptable types are:

- station line,
- profile grade line,
- inside or outside edge of pavement, or centerline of roadway.

Stationing should run left to right. Exceptions may be necessary, but in all cases orient the structures in a group similarly regardless of stationing.

6.1.2. Elevation

Place the elevation view directly aligned above the plan view. Project the elevation view vertically from the lower side of the plan view.

Use the same scale as the plan view.

Show a datum line with stationing. Select a datum elevation that is a multiple of 10 feet (e.g. 0.00, 1450.00, or 100.00). Additionally, select a datum elevation so the datum line is a sufficient distance below the drawing with no part of the elevation view touching the datum line.

6.1.3. Profile Grade

Place the profile grade line diagram above the elevation view.

The preferred scale is 1"=10'-0" but the scale may be exaggerated horizontally or vertically to accommodate all relevant information, i.e. "No Scale" is used.

6.1.4. Typical Section

Place the typical section in upper right-hand portion of sheet.

The preferred scale is 1/4"=1'-0". The scale may be adjusted, but for large structures, place a detail on the GENERAL LAYOUT sheet drawn at a small enough scale so the entire structure fits. Then use the TYPICAL SECTION sheet for the structure drawn at a more detailed scale.

Show the section looking ahead on station. Identify by section letters or stationing if the section varies or station lines are not continuous across the structure.

Show pier(s) for multi-span bridges. Do not show abutments on single-span bridges.

6.1.5. Sheet Title

Include the official name and number of the structure in the title block. See Section 10.6. for Department bridge identification standards. Only use abbreviations when necessary.

6.1.6. Miscellaneous

Keep the GENERAL LAYOUT sheet neat and clean with clear, legible lettering. This sheet is subject to review by other agencies.

Provide an area on the GENERAL LAYOUT about 6 inches square in the lower right-hand corner for the

Drawing Index and Estimate of Quantities, if possible. The Drawing Index includes all applicable bridge sheets and the TEST BORING LOG AND LOCATIONS sheets from the foundation engineer.

In the lower right-hand corner include a symbol and note for the location of the bridge number plates and minimum vertical clearance, if applicable.

The stations and elevations on this sheet should be shown to no more than two decimal points.

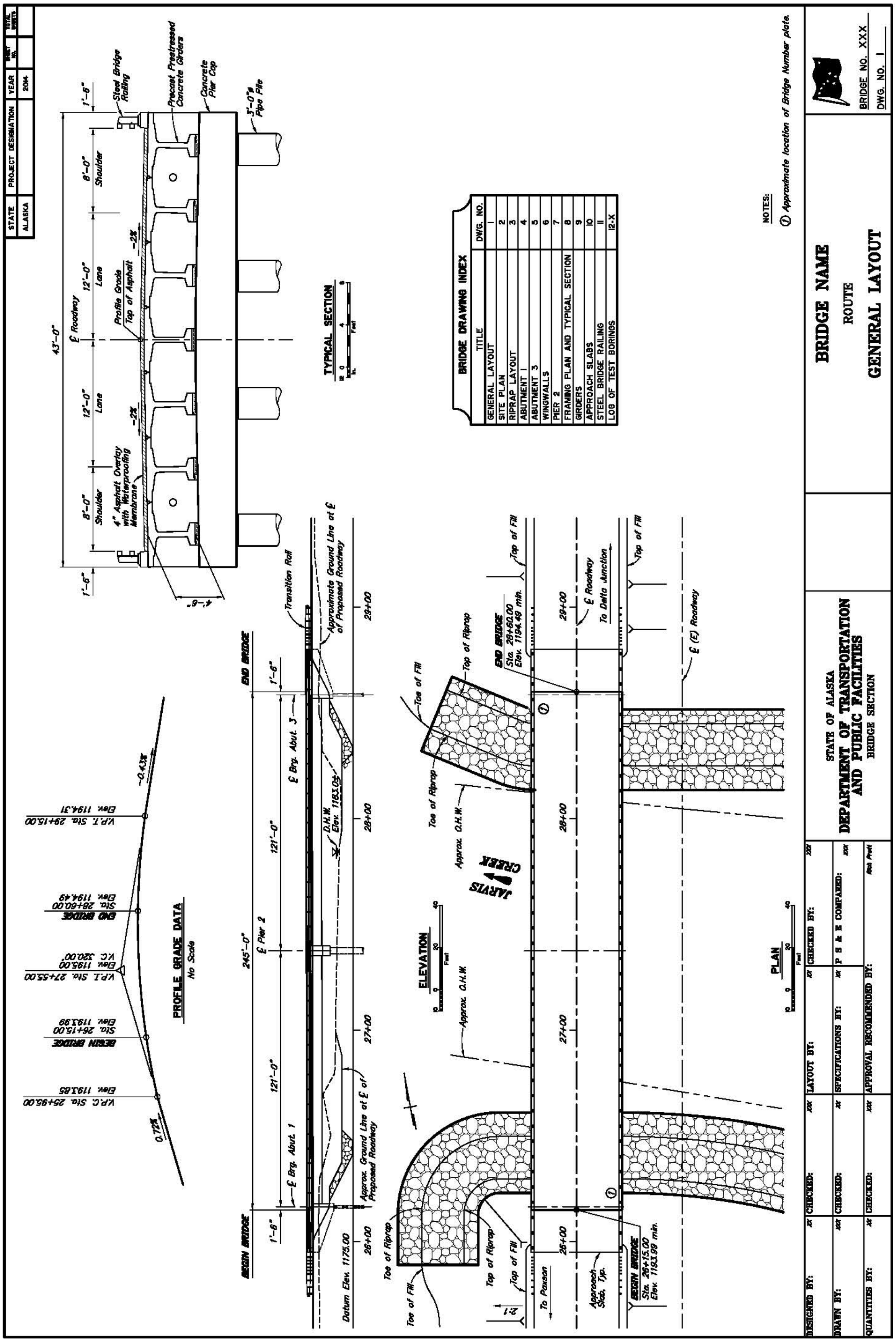

Figure 6-1
Example General Layout Sheet

6.2. Site Plan Sheet

6.2.1. General Notes

Place the General Notes in the upper half of the sheet toward the right-hand side. An example of the General Notes content and format is shown in Figure 6-2.

6.2.2. Site Plan

Place the site plan in the upper, left-hand side of the sheet. The site plan scale is typically the same as the scale for the plan view on the GENERAL LAYOUT sheet. Show alignment of the centerline, centerline of bearing with alignment, right-of-way lines, and utilities.

6.2.3. Miscellaneous

Place the foundation table (e.g. Pile Data Table) below the General Notes, and place the abbreviations list below the foundation table. The lower left-hand side of the sheet can be used for miscellaneous items, such as the Estimate of Quantities table if sufficient space is not available on the GENERAL LAYOUT sheet.

GENERAL NOTES

DESIGN: ....AASHTO LRFD Bridge Design Specifications, Fifth Edition, 2010, with latest interim specifications. Seismic design per AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2009.

LIVE LOAD:.............................HL-93

DEAD LOAD:............................Includes 50 psf for all wearing surfaces.

SEISMIC PARAMETERS:...........PGA = 0.54
S_s = 1.18
S_1 = 0.46
Site Class = D
Liquefaction Potential = Low
AASHTO 7% probability of exceedance in 75 years.

REINFORCEMENT:.....................ASTM A706, Grade 60, Fy = 60,000 psi
Space reinforcement evenly unless otherwise noted.
ASTM A970 Class HA Headed bars.

PRESTRESSED CONCRETE:.......See "GIRDERS" Dwgs.

CONCRETE:.............................Class A Concrete, f'c = 4000 psi, unless otherwise noted.

STRUCTURAL STEEL:...............ASTM A709 GR36T3, Fy = 36,000 psi, unless otherwise noted. Galvanize all structural steel in accordance with AASTHO M111 unless shown otherwise.

STRUCTURAL STEEL PILING:..H-Piles – ASTM A709, GR50T3, Fy = 50,000 psi.
Pipe Piles – API 5L X52 PSL2, Fy = 52,000 psi.
Pile Tip reinforcing is required.

Figure 6-2
Typical General Notes

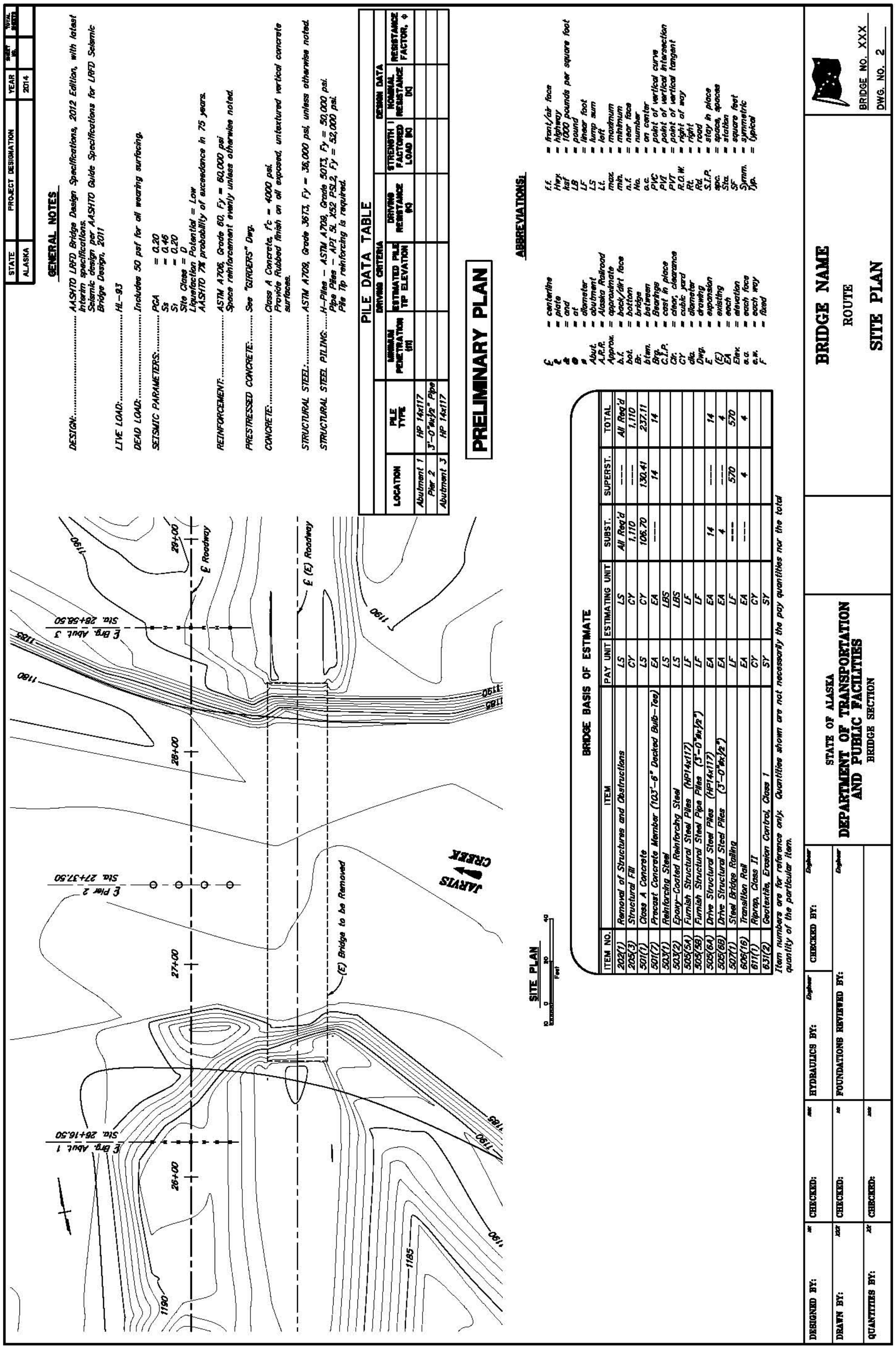

Figure 6-3
Example Site Plan Sheet

6.3. Abutment Sheet

6.3.1. Plan

Place at the plan view at the top, left side of sheet.

The preferred scale is 1/4”=1’-0”. Use a scale of 1/8”=1’-0” for large structures, but show less detail.

Show partial reinforcement to indicate orientation of rebar on skewed bridges.

6.3.2. Elevation

Align the elevation view under the plan view on the left side of sheet. Include a note if the view is looking back on station when showing Abutment 1.

Use the same scale as the plan view.

Do not show rear elevations.

Show utiliducts with a solid line and girders with a DOT2 linetype on PEN3.

6.3.3. Sections and Details

The preferred scale for sections and details is 1/2”=1’-0” minimum. Do not show sections and details with reinforcement at less than a 3/8”=1’-0” scale.

Orient sections in the direction of the view or section cut.

Align the bottom of the footing with the bottom of the footing from the elevation view.

Sections and details are to be taken from the plan, elevation, or secondary views rather than from other sections.

Do not repeat details in similar sections.

Do not shade or show aggregate in sections.

Label mandatory construction joints.

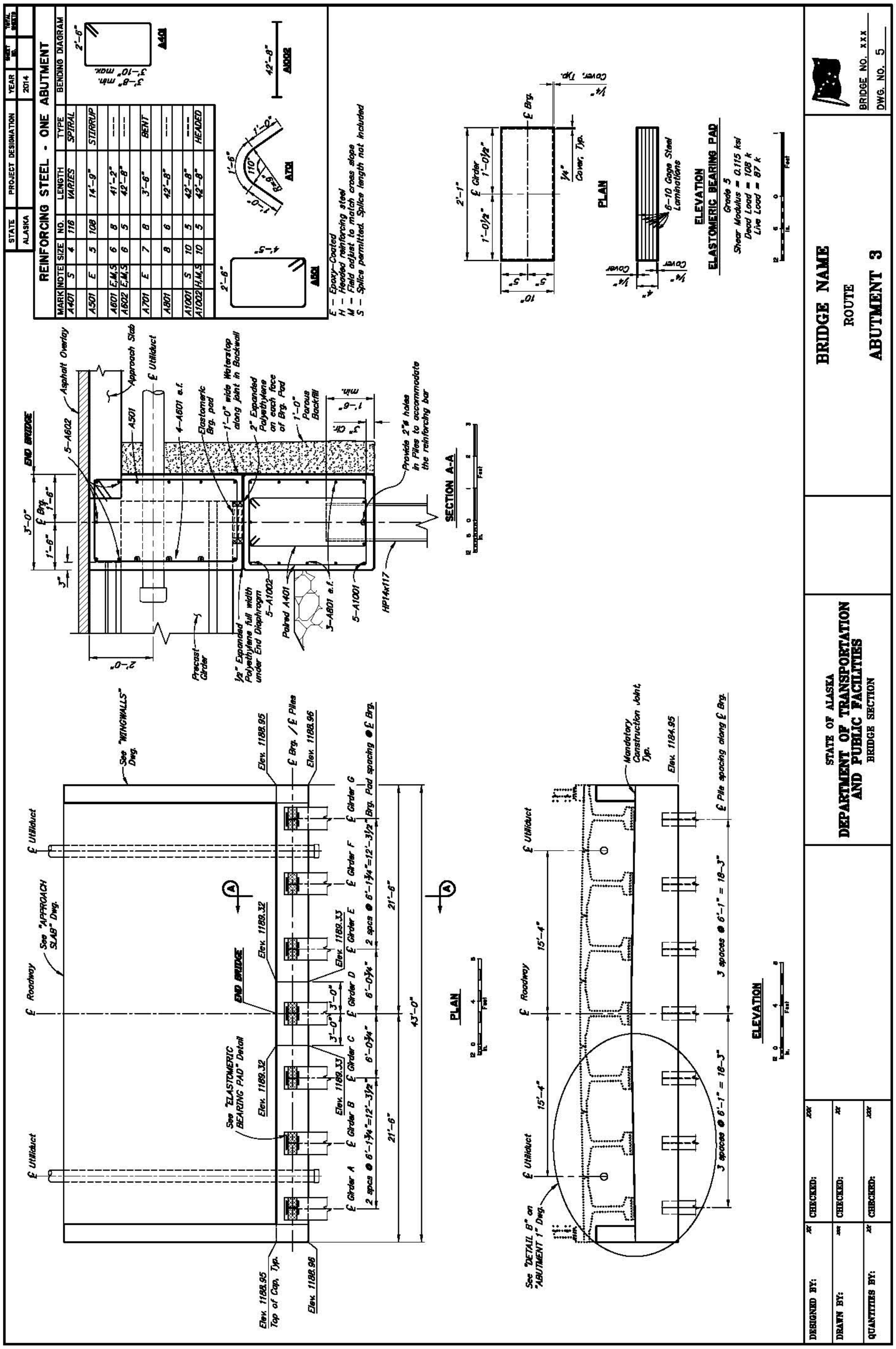

Figure 6-4
Example Abutment Sheet

6.4. Wingwall Sheet

6.4.1. Elevation

Place the elevation view at the top left side of the sheet.

The preferred scale is 1/2"=1'-0". Use a scale of 1/4"=1'-0" for large structures.

Show the elevation looking normal to outside face of the wingwall.

6.4.2. Sections

Use projections for sections of the elevation view.

Use the same scale as the elevation view.

Label rebar without spacing.

Do not show abutment reinforcement.

Do not show sections and details with reinforcement less than a 3/8"=1'-0" scale; the preferred scale is 1/2"=1' minimum.

Orient sections in the direction of the view or section cut.

Do not shade or show aggregate in sections.

6.4.3. Finished Elevation

Use 3/8"=1'-0" scale minimum; the preferred scale is 1/2"=1'-0".

Orient the finished elevation view in the same direction as the elevation view.

Show the finished ground line with earth hatching angled at 45 degrees on PEN6.

Use layer PEN5 for railing, curb, and girders.

Use the Hidden linetype for everything below the finished ground line.

Do not show rebar in this view.

Do not show the abutment outline hidden behind the wingwall.

Label the finished ground line.

Show the expansion joint in the concrete curb and steel rail or concrete barrier, whichever applies.

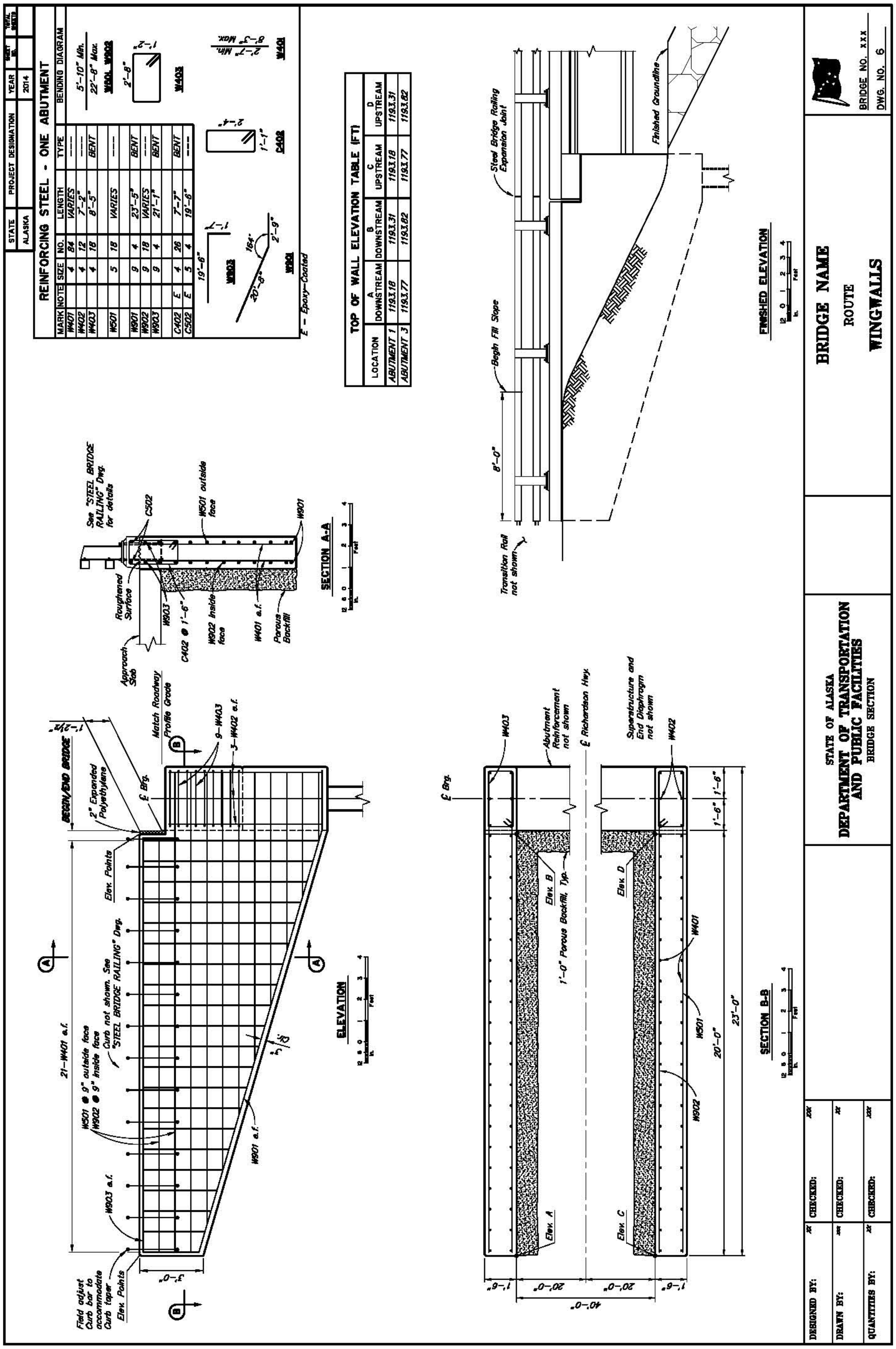

Figure 6-5
Example Wingwall Sheet

6.5. Pier Sheet

6.5.1. Plan

Place the plan view at the top left side of sheet.

The preferred scale is 1/4”=1’-0”. Use a scale of 1/8”=1’-0” on large structures.

Do not show layout information that is shown on the site plan.

Show partial reinforcement to indicate the orientation of rebar on skewed bridges.

6.5.2. Elevation

Use projections for the elevation from Plan view.

Use the same scale as the plan view.

Show girders when spacing allows.

6.5.3. Sections

Use a minimum scale of 3/8”=1’-0”; the preferred scale is 1/2”=1-0”.

Orient sections in the direction of the section cut.

Call out reinforcing steel clearances other than 2”.

Architectural columns require multiple section cuts.

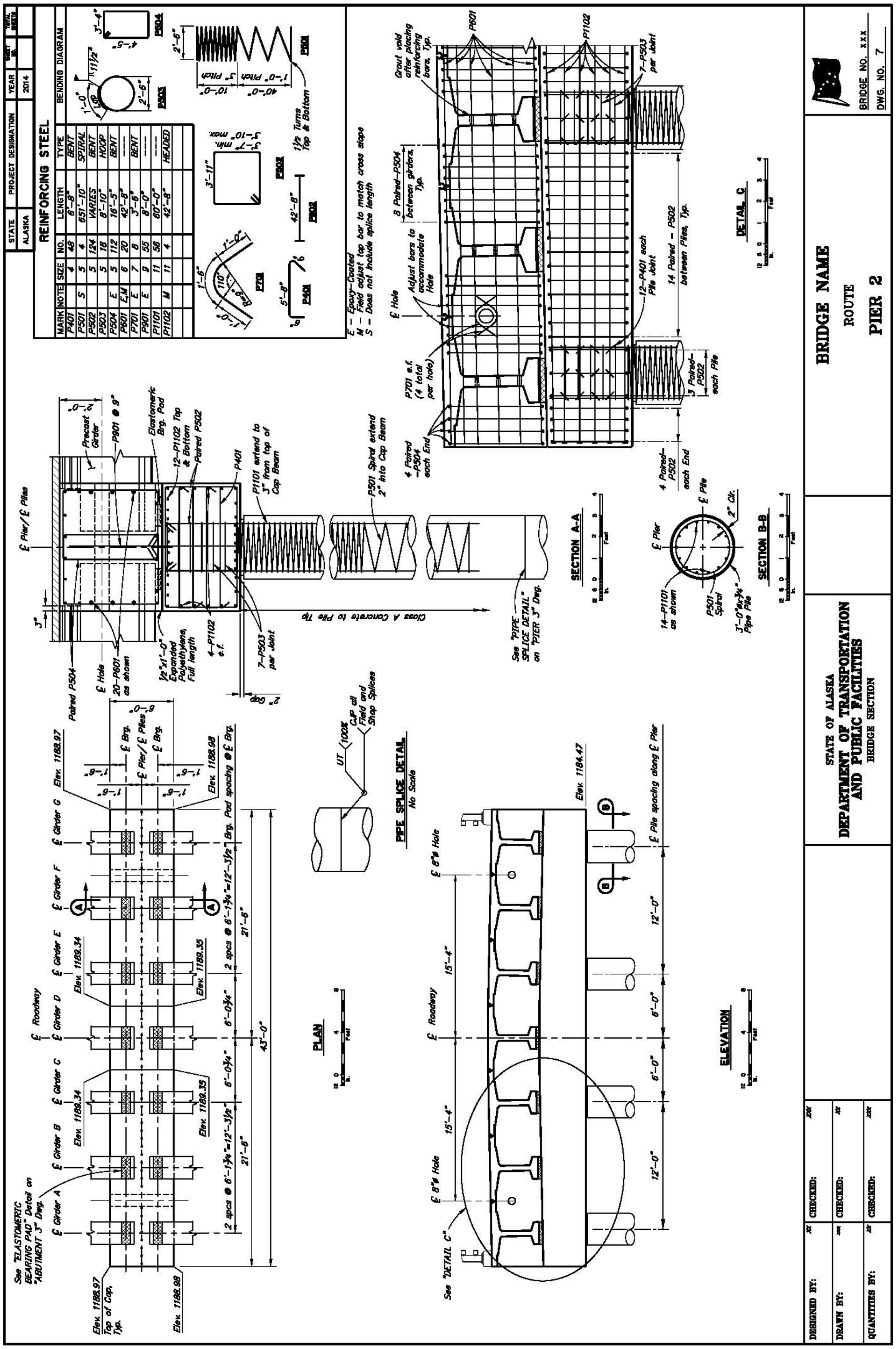

Figure 6-6
Example Pier Sheet

6.6. Framing Plan and Typical Section Sheet

The Typical Section and Framing Plan may be placed on separate sheets if necessary.

6.6.1. Typical Section

Place the typical section at the top, left side of sheet looking ahead on station.

The preferred scale is 1/4"=1'-0". Use a 1/8"=1'-0" scale on large structures.

Show concrete diaphragm reinforcement.

Show dimensions for overall structure and centerline utilities.

6.6.2. Framing Plan

Place the framing plan on the bottom half of the sheet. Orient the framing plan on the sheet the same way as the plan view on the GENERAL LAYOUT sheet.

Use a scale that shows the bridge length with approach slabs.

Do not show stationing.

Show dimensioning for the overall bridge length, shear key spacing, bridge railing for payment, bridge railing post spacing, centerline girder spacing from edge of deck and centerline of structure, and diaphragm spacing.

Label each centerline girder (e.g. Girder A, Girder B, etc.). Show the roadway centerline with Center linetype on layer PEN5. Show the centerline of bearings with Center2 linetype on layer PEN5. Show the centerline of girders with Center2 linetype on layer PEN5.

6.6.3. Diaphragm Sections and Details

Use a scale of 3/4"=1'-0" for sections and details with reinforcement.

Orient sections and details in the direction of the view or section cut.

Do not shade or show aggregate in sections.

Show dimensions related to the diaphragm.

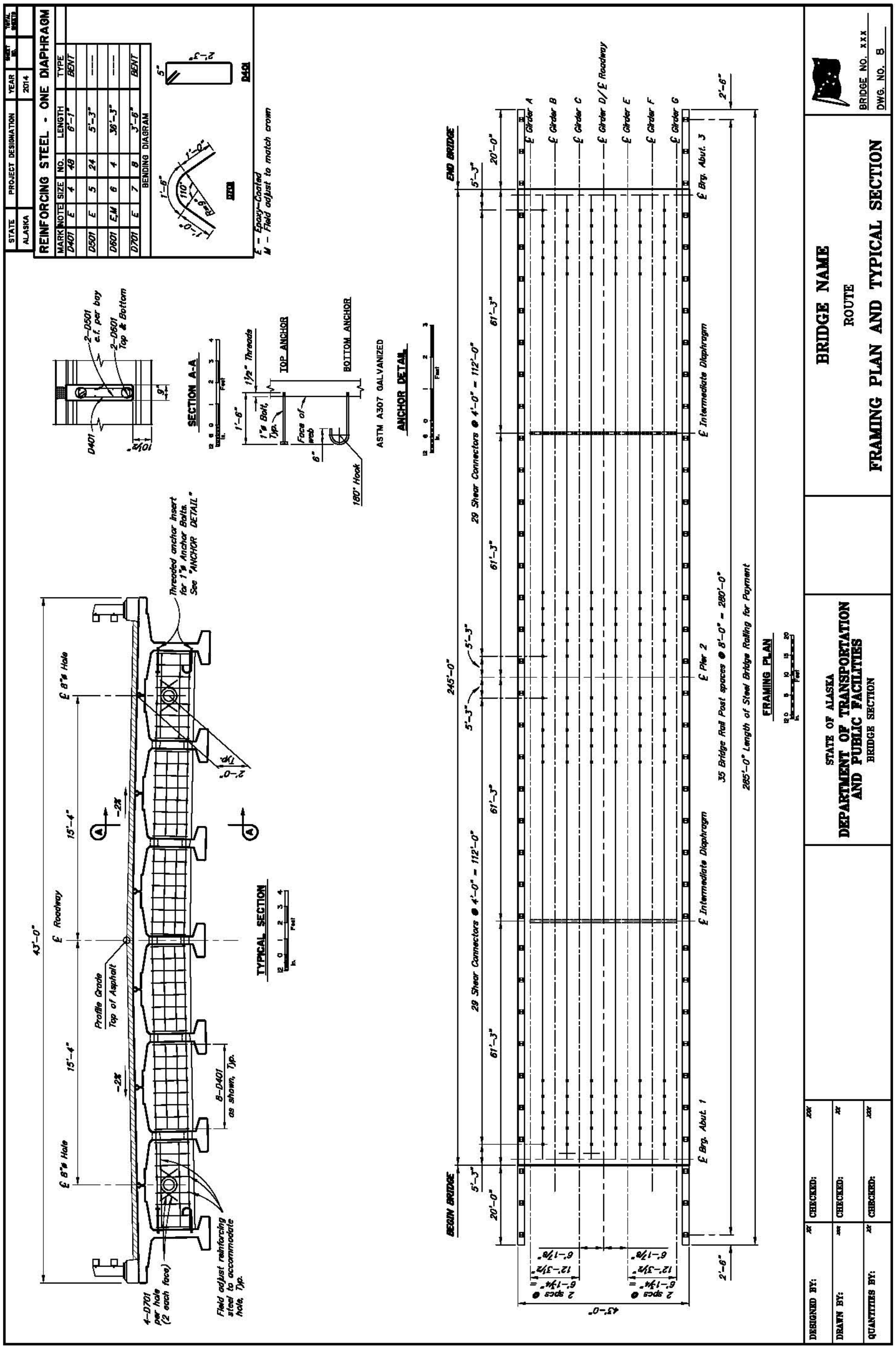

Figure 6-7
Example Framing Plan and Typical Section Sheet

6.7. Girder Sheet

This section pertains to precast, prestressed concrete girders. Steel girder designs are less common, so refer to the most recent steel bridge plan set for drawing layout and drafting.

6.7.1. Plan

Place the plan view at the top, left side of sheet looking ahead on station.

Draw the plan view to scale, but due to the areas removed by the break lines show as "No Scale". Include the ends and center portion of the girder in the plan view. Show either the exterior or interior top flange reinforcing steel spacing; do not show both. Include enough of the reinforcing steel at the ends of the deck to include the first shear key.

6.7.2. Elevation

Place the elevation view below the plan view.

Draw the elevation view to scale, but similar to the plan view show "No Scale" due to the break lines. Ensure that each section of shear reinforcing steel (typically G403 bars) spacing is included in the elevation view.

Section the elevation view at the bearing and near the midspan. Show these sections and an exterior midspan section below the elevation view. None of these views have a scale.

6.7.3. Girder Notes and Details

Place the Reinforcing Steel Schedule in the upper right-hand corner.

Place the Girder Notes under the reinforcing steel schedule.

Place the shear key detail, shear connector detail, and associated view as space allows on the rest of the drawing.

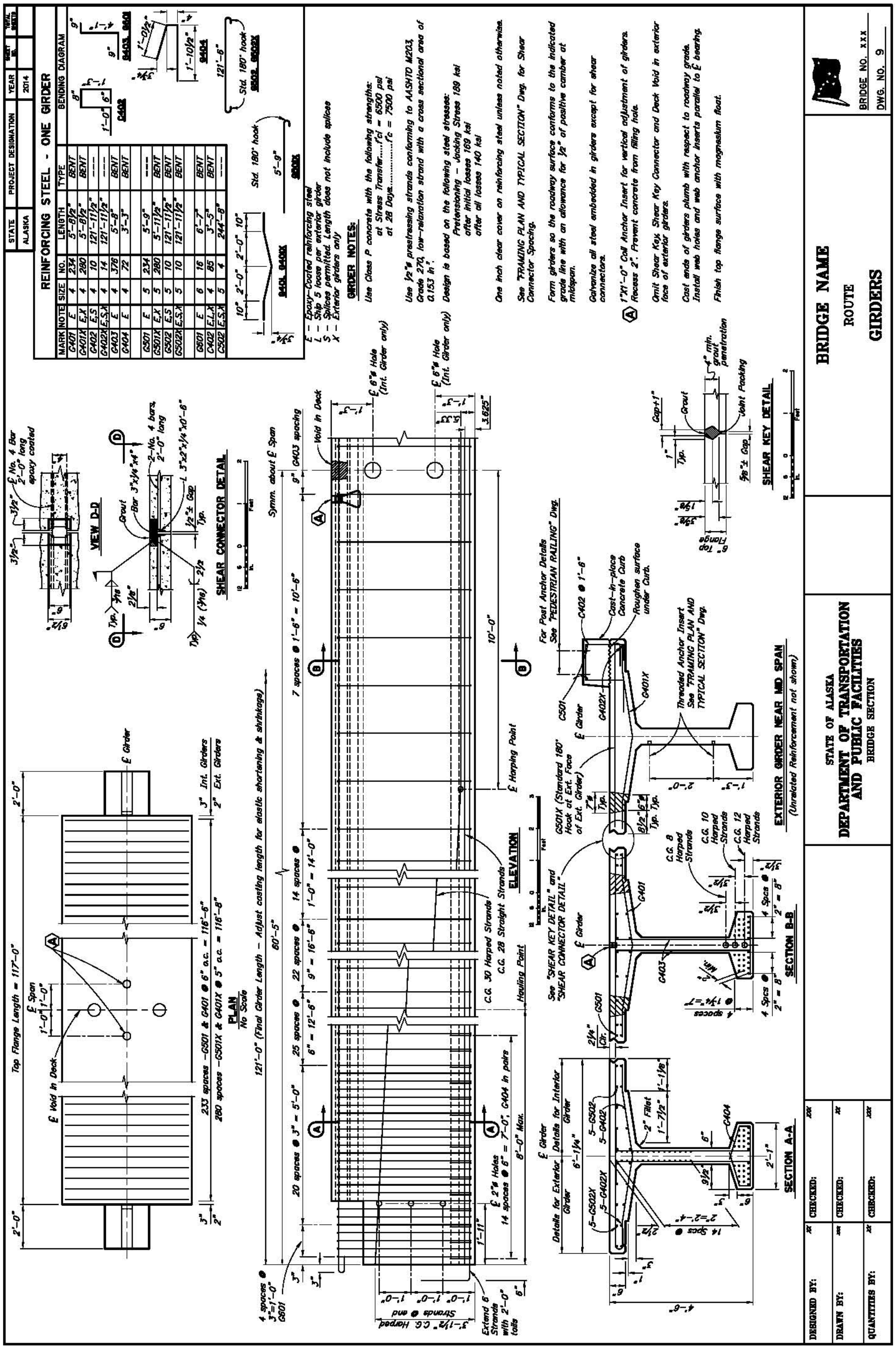

Figure 6-8
Example Girder Sheet

Appendix 6.A

PLAN SHEET CHECKLISTS

Appendix 6.A presents the following standardized checklists for each of the following most common plan drawings:

- General Layout;
- Site Plan;
- Abutment;
- Wingwall;
- Pier;
- Framing Plan and Typical Section; and
- Girder.

Project No. ______________________________ Date: ___________________________________

Bridge No. ______________________________ Designer(s): ______________________________

GENERAL LAYOUT

Are the following items properly included on the GENERAL LAYOUT sheet?	**Yes**	**No**	**N/A**
1. Plan • North arrow • Traveled way, shoulder and median width of approach roadways • Top and toe of approach fill or cut • Slope of fill or cut • Horizontal clearance under structure (including future alignment) • Alignment data • Name and direction of nearest towns or cities • Designation of construction and/or bridge centerlines • Skew angle (between the normal or radial to centerline of structure and centerline of pier or abutment) • Location of minimum vertical clearance • Deck drains and manholes • Location of bridge number plates • Begin Bridge (BB) and End Bridge (EB) station and elevation • Bank protection or slope paving • Centerline piers or bents • Structure mounted signs • Temporary railings and approach rail • Tie between toe of slope and railroad tracks • Railroad right-of-way lines for structures over railroad tracks • Approach slab	☐	☐	☐
2. Elevation • Abutment and pier numbers • Datum line with elevation and stations • Original ground line at bridge centerline, or as noted • Total length of bridge (BB to EB) • Span lengths • Bank protection or slope paving • Vertical clearance to nearest 1” (round off to lower number)	☐	☐	☐
3. Profile Grade • Gradients, pertinent elevations and stations • Show length of vertical curves. • Do not show superelevation diagram on GENERAL LAYOUT sheet	☐	☐	☐

Are the following items properly included on the GENERAL LAYOUT sheet?	Yes	No	N/A
1. Typical Section • Width of traveled way, sidewalks, shoulders, and medians on bridge • Overall width of bridge • Location of profile grade • Crown or maximum superelevation • Surfacing and waterproofing membrane • Superstructure thickness top of deck to bottom of girder (or main slab) • Type of girder (e.g. steel, composite, non-composite, precast prestressed, or cast-in-place prestressed, etc.) • Utilities and openings for future utilities • Typical pier for multiple spans or no substructure for single spans • Rail or barrier type	☐	☐	☐

Project No. ______________________________ Date: ___________________________________

Bridge No. ______________________________ Designer(s): ____________________________

SITE PLAN

Are the following items properly included on the SITE PLAN sheet?	**Yes**	**No**	**N/A**
1. Site Plan • Remove contour lines that interfere with dimensions, stations, or bearings. • Relocate any interfering pertinent information, especially utility data and existing features to be removed. • Stations and bearing of centerline piers and abutments. • If all supports are parallel, the bearings may be noted accordingly. • Same layout line on the Site Plan as used on other details. • Sufficient layout information for a survey crew to stake out hubs for the contractor's use • Horizontal and vertical data for Bench Mark (if present) near bridge • Do not repeat layout information on details sheets, particularly bearings, stations, and curve data. • Pile layouts if not convenient to show elsewhere in the plans • Outline of existing bridge "To Be Removed" • Riprap dimensions and layout line geometry, if not shown on a RIPRAP LAYOUT sheet. • Ordinary High Water Line	☐	☐	☐
2. General Notes	☐	☐	☐
3. Foundation Table	☐	☐	☐
4. Hydraulic Summary, if not shown on a RIPRAP LAYOUT sheet.	☐	☐	☐

Project No. ______________________________ Date: ________________________________

Bridge No. ______________________________ Designer(s): ____________________________

ABUTMENT SHEET

Are the following items properly included on the ABUTMENT sheet?	Yes	No	N/A
1. Plan • Dimension along same layout line as is stationed on Site Plan • Do not dimension piles from edges of footings • Layout dimensions only; other dimensions shown on larger details • Centerline of roadway • Centerline of bearings • Bearing pads with hatching • Begin/End Bridge	☐	☐	☐
2. Elevation • Piles and dimensions • Girders, curb, and railing with DOT2 linetype on PEN3 layer • Avoid detail dimensions • Section cut lines • Utility holes • Weep holes • Show elevations on slope paving; exception, a constant dimension below soffit	☐	☐	☐
3. Section Views	☐	☐	☐
4. Bearing Pad Details	☐	☐	☐
5. Reinforcing Steel Table (“Bar Chart”)	☐	☐	☐

Project No. ______________________________ Date: ___________________________________

Bridge No. ______________________________ Designer(s): ____________________________

WINGWALL SHEET

Are the following items properly included on the WINGWALL sheet?	**Yes**	**No**	**N/A**
1. Elevation • Railing not shown • Section cuts	☐	☐	☐
2. Sections • Do not show dimensions; dimensions shown in plan view. • Utilities not shown • Centerline of roadway and bearing	☐	☐	☐

Project No. ______________________________ Date: __________________________________

Bridge No. ______________________________ Designer(s): ____________________________

PIER SHEET

Are the following items properly included on the PIER sheet?	**Yes**	**No**	**N/A**
1. Plan • Railing not shown • Dimension along same layout line as stationed on the SITE PLAN • Centerline of roadway • Centerline of bearings • Bearing pads shown with hatching	☐	☐	☐
2. Elevation • Geometry dimensions for pier cap and column • Utility openings • Centerline of roadway. • Crain and drain pipe, if required • All footings • Column bars and stirrups or spiral reinforcing • Cap stirrups and reinforcement • A few footing reinforcing bars shown to identify location of bars in mat • Location of sections • Footing elevations	☐	☐	☐
3. Section Views	☐	☐	☐
4. Bearing Pad Details (if not shown on ABUTMENT sheet)	☐	☐	☐
5. Reinforcing Steel Table (“Bar Chart”)	☐	☐	☐

Project No. ______________________________ Date: ___________________________________

Bridge No. ______________________________ Designer(s): _____________________________

FRAMING PLAN AND TYPICAL SECTION

Are the following items properly included on the FRAMING PLAN AND TYPICAL SECTION sheet?	**Yes**	**No**	**N/A**
1. Typical Section • Utilities with reinforcement around opening • Asphalt overlay • Section cut for diaphragms • Anchor bars for diaphragms • Substructure not shown • Lane and shoulder dimensions not shown	☐	☐	☐
2. Framing Plan • Approach slab • Rail posts • Shear keys • Diaphragms with Hidden2 linetype on PEN1 • Dimension out to out of deck	☐	☐	☐
3. Diaphragm Details	☐	☐	☐

Project No. ______________________________ Date: ________________________________

Bridge No. ______________________________ Designer(s): ____________________________

GIRDER

Are the following items properly included on the GIRDER sheet?	**Yes**	**No**	**N/A**
1. Plan • Deck reinforcing steel spacing • Void locations	☐	☐	☐
2. Elevations • Reinforcing stirrup spacing • Final girder length • Harping point • Hauling point • Holes, center and at ends • Harped and straight strand center of gravity	☐	☐	☐
3. Sections • Girder geometry • Prestressing steel number and locations • Reinforcing steel types and locations • Voids • Exterior girder inserts	☐	☐	☐
4. Girder Notes	☐	☐	☐
5. Reinforcing Steel Table (“Bar Chart”)	☐	☐	☐

7. Construction Contracts

The Bridge Section performs specific support activities for contract advertising and award when structural items are a part of a contract. See the *Alaska Highway Preconstruction Manual* (Section 470) for more discussion on advertising and award.

7.1. Contract Documents

The *Alaska Department of Transportation and Public Facilities Standard Specifications for Highway Construction* (*Alaska Standard Specifications*), *Standard Modifications* and *Special Provisions* to the Standard Specifications, Plans, and *Standard Drawings* are all essential elements of a contract. They are intended to complement each other and are used to provide complete instructions for the work to be accomplished.

If a discrepancy exists among these documents, the order of precedence is:

1. Special Provisions
2. Plans
3. Standard Modifications
4. Standard Specifications
5. Standard Drawings

All bridge related construction specifications must comply with the documents referenced in 23 CFR 625.4 as minimum standards. When the *Alaska Standard Specifications* are more strict than the AASHTO LRFD Construction Specifications, the *Alaska Standard Specifications* take precedence.

7.1.1. Standard Specifications for Highway Construction

Statewide Design & Engineering Services (D&ES) is responsible for the *Alaska Standard Specifications*, which present the construction requirements and materials to construct highway, traffic, and bridge projects.

7.1.2. Standard Modifications

D&ES is responsible for the *Standard Modifications to the Standard Specifications for Highway Construction* (Standard Modifications), which update the requirements set forth in the latest version of the *Alaska Standard Specifications*. Standard Modifications cannot be revised once they have been adopted by DOT&PF.

7.1.3. Special Provisions

Special Provisions to the Standard Specifications for Highway Construction (Special Provisions) are additions or revisions to the *Alaska Standard Specifications* setting forth conditions and requirements on a specific project.

Special Provisions, which are prepared by the bridge designer, are included in the contract documents for that project and are not intended for general use. Section 7.2 discusses guidelines for preparing Special Provisions.

Special Provision Types

The two basic types of Special Provision presentations are performance-based and material or method-based.

The performance-based specification that describes the end result of construction is the preferred type. The procedures and resources to achieve the end result are at the Contractor's discretion.

The material or method-based presentation (usually referred to as "prescriptive specifications") describes in detail the procedure and materials that should be used to construct the element.

7.1.4. Standard Drawings

Bridge Standard Drawings

The Bridges and Structures Standard Drawings provide details on various bridge and transportation structural elements that are consistent from project-to-project (e.g., retaining walls, bridge rail to guardrail transitions). The *Standard Drawings* are available from the Department's website. The Bridge Section updates the bridge standard drawings as needed.

Standard Drawings

The *Standard Drawings* provide road and traffic details for various design elements that are consistent from project-to-project (e.g., guardrail, sign posts, fencing, drainage appurtenances).

The *Standard Drawings* are available from the Department's website.

7.2. Special Provision Preparation

Special Provisions are required when a project contains work, material, a sequence of operations, or any other requirements necessary for the completion of the project but not addressed in the construction plans, *Alaska Standard Specifications* or Standard Modifications.

Write the Special Provision so that the prospective bidder can clearly understand the work, materials and construction requirements, how the item of work will be measured, and the basis of payment.

Use the following steps when preparing a Special Provision:

Define Need

Review the *Alaska Standard Specifications*, Standard Modifications, *Standard Drawings*, and construction plans to ensure the Special Provision is needed. Prepare a Special Provision only if the topic is not adequately covered in one of the other contract documents.

Research

Research the topic so that complete and detailed information is available before writing the Special Provision. This may require contacting manufacturers, contractors, or suppliers for the latest information. Local conditions and problems should also be fully investigated.

Type of Special Provision

Analyze the type of construction to be covered in the Special Provision to determine whether a performance-based or material or method based Special Provision is needed.

The performance-based specification is the preferred type of Special Provision. Under some circumstances, both types may be necessary within the compiled Special Provisions, but do not mix the presentation types within an individual section of the Special Provision.

Develop Outline

Outline the basic work and material requirements. Organize all relevant factors under each appropriate heading.

Writing the Special Provision

Write the Special Provision in the active voice and the imperative mood (sentence expresses a command). "Active voice" is when the subject of the sentence is performing the action; "passive voice" is when the subject of the sentence is the receiver of the action. For example:

- Use Active Voice: "Apply rubbed finish to exposed surface."
- Avoid Passive Voice: "Rubbed finish shall be applied to exposed surface."

<u>Sentences</u>. Prepare the Special Provision using simple language and words. Strive to keep words and sentences short (20 words or less).

<u>Paragraphs</u>. Limit paragraphs to three to four sentences.

<u>Terminology</u>. Use words consistent with their exact meaning. Use the same word throughout; do not use synonyms. Avoid any words that have a dual meaning. Omit extraneous words and phrases.

<u>Pronouns</u>. Avoid the use of pronouns, even if frequent repetition of nouns is necessary.

<u>Punctuation</u>. Carefully consider the punctuation using the minimum number of punctuation marks consistent with the precise meaning of the language. Ensure that there can be no doubt on the meaning of any sentence.

<u>Parentheses</u>. Avoid the use of parentheses. Instead, use commas or rewrite the sentence.

<u>Numbers</u>. It is usually unnecessary to write numbers both in words and figures. For example, do not write "Use four (4) 1-in bolts." Instead, write "Use four 1-inch bolts."

When writing dimensions, always use numerals (e.g., 2.0 inch, 10 feet, 20 cubic yards). Write "2 inch by 4 inch" not "2 in × 4 in."

Times and dates should be written numerically. Write fractions as decimals. Decimals less than one should be preceded by the zero (e.g., 0.25 inches).

Reviewing

Review previously completed paragraphs as succeeding ones take shape. Where necessary, redraft preceding paragraphs to reflect later thoughts.

7.2.1. Format

Prepare Special Provisions in the same format as the *Alaska Standard Specifications*. Organize the subsections in this order:

1. Description

2. Materials
3. Construction Requirements
4. Method of Measurement
5. Basis of Payment

Description
Describe the work, with references to the *Alaska Standard Specifications*, plans, or other Special Provisions that further define the work. Where necessary for clarity, describe the relationship of this work item to other work items or other phases of construction.

Materials
Designate the materials used in the work item. Reference, in this order of preference, the Alaska Test Methods (ATM), AASHTO, ASTM, or other recognized specifications when possible. If an existing reference does not adequately describe the material properties, a Special Provision in *Alaska Standard Specifications* Section 700 may be necessary to delineate complete specifications of the properties of each material and the method of tests.

Construction Requirements
Describe the sequence of construction operations or the desired end product. Do not mix performance-based and method-based specifications. Where practical, use only the performance-based presentation. Only use the method-based presentation for the sequence of construction operations if performance-based language would not achieve the desired result. Specify quality control and construction tolerances.

Method of Measurement
Describe the components of the completed work item to be measured for payment, and provide the units of measurement. If the pay unit differs from the defined units in *Alaska Standard Specifications* Section 109, then designate any supplemental requirements needed to establish a definite measured unit, such as when and where to make measurements, how to address waste, and what is excluded from measurement.

Basis of Payment
Describe payment units and define the scope of work covered by such payment.

7.2.2. Guidelines

Ensure that the Special Provision satisfies the elements of a clearly written and authoritative document. It must be complete, clear, concise, correct, and consistent.

Complete
Ensure the essentials are included and that each requirement is definitive and complete.

Clear
Clearly delineate the method of measurement and the basis of payment, including any subsidiary items.

Clearly describe the job requirements for general conditions, types of construction and quality of workmanship. Do not leave the bidder in doubt about work requirements. Never conceal difficulties or hazards from the Contractor.

Avoid conflicting or ambiguous requirements. Every specification should have only one meaning.

Do not use phrases such as “as approved by the Engineer,” “at the discretion of the Engineer,” or “as directed by the Engineer” in place of definite workmanship requirements. These types of phrases may lead to confusion or misunderstanding. The Contractor cannot anticipate what the Engineer will want.

Give directions, never suggestions.

Concise
Write each Special Provision as concisely as practical. When reviewing the Special Provision, consider the following suggestions:

- Avoid duplications between the Special Provision and any related contract documents. Do not repeat any instruction, requirement, direction, or information given elsewhere in the contract documents.
- Do not give reasons for a specification requirement.
- Do not provide information that is unnecessary for bid preparation and accomplishing the work.
- Minimize the use of cross-references.

Correct
Ensure the Special Provision is written accurately. Where practical, independently check every factual statement. Do not include items that cannot be required or enforced. Consider the practical limits of

workers and materials to avoid specifying impossibilities.

Ensure that the specification does not unintentionally exclude an acceptable product, construction method, or equipment. Ensure the specification does not change the basic item design.

Specify standard sizes and dimensions wherever practical.

Specify material durability or reliability requirements. When possible, use permanent and recognized standards to ensure specified performance or characteristics are achieved. If not, completely and accurately define the testing criteria.

Make a careful, critical examination of manufacturers' or trade associations' recommendations, and require supporting evidence before adopting them. Do not specify a sole source or show a preference for a specific product without an approved public interest finding.

Keep requirements stringent. A stringent requirement can be relaxed if the need arises; however, adding requirements after the contract has been awarded may increase cost.

Consistent
Give directions in the Special Provision that are consistent with current DOT&PF standard practice.

Ensure the Special Provision is consistent in language selection, usage, format, and organization with the *Alaska Standard Specifications*.

7.2.3. Terminology

Abbreviations: Generally, avoid abbreviations; however, abbreviations may be used if they are defined and the definitions are consistent with the accepted meanings.

Amount, quantity: Use "amount" when writing about money only. When writing about measures of volume (e.g., yd^3, gallons), use "quantity."

And/or: Avoid using "and/or"; instead, use "and" alone, or "or" alone, or "or ... or both." For example, "Unless otherwise shown on the plans or specified in the Special Provisions or both, ..."

Any, all: The word "any" implies a choice and may cause confusion. Use the term "all" in place of "any." For example, "Repair all defects."

As per: Do not use "as per"; instead, use "as stated," "as shown," "conforming to" or other similar phrases.

At the Contractor's expense: Do not use the phrase "at the Contractor's expense"; instead use, "at no cost to Department" or "subsidiary to the cost of other contract items."

Balance, remainder: Use the term "balance" when referring to money. Use "remainder" to describe something or material left over.

Coarse, course: Use "coarse" to describe textures and "course" for layers.

Conform: Use the word "conform" to refer to dimensions, sizes and fits that must be strictly adhered to (e.g., "cut bolt threads conforming to ASA Standards, Class 2 fit, coarse thread series"). Where a better product is acceptable, use the phrase "meeting the requirements of..."

Contractor: Use the word "Contractor" in place of the word "bidder" when writing Special Provisions for construction. Only use "bidder" for proposals.

Approved Equal: Do not use the phrase "approved equal." The Contractor may not know what is truly equivalent before awarded the contract.

Proposal: Do not use the word "proposal" when the word "contract" is intended. Only use the term "proposal" to describe requirements during the bidding process.

Resisting, resistant: Do not use "corrosion-resisting," but instead use "corrosion-resistant."

Said: Do not use "said pipe" or "said aggregates" but, instead, use "this pipe" or "these aggregates."

Shall: Avoid using the word "shall." DOT&PF has adopted "active voice" specifications. Rewrite the sentence using the active voice.

Such: Do not end a sentence with the word "such." "Such" usually means "of this or that kind," or similar to something stated. Instead, state what is actually meant or name the work to be completed, or rephrase the sentence.

Symbols: Do not use the following symbols when writing Special Provisions:

Symbol	Write Instead
/	per, or "a"
%	percent

+	plus
–	minus
×	by

The: Do not eliminate "the" for brevity.

Thoroughly: Avoid using the adverb "thoroughly," as in thoroughly wet, thoroughly dry, thoroughly clean, etc., because it is unenforceable. Preferably, state the value of the intended requirements in percent, dimensions, number of passes, etc.

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8. Quantity Calculations and Cost Estimates

The Bridge Section supports the project manager by providing several types of quantity calculations and cost estimates for structures during the planning, environmental, and design stages. These submissions are needed for planning and budgeting purposes.

8.1. Planning Stage (Level 1)

Bridge Section frequently provides planning estimates for projects involving bridges and other transportation structures.

Typically planning level estimates are requested at the Statewide Transportation Improvement Program (STIP) development and regional project initiation stage. These estimates are developed quickly, with very limited information, and require many assumptions. The following applies to developing a planning cost estimate:

8.1.8. Responsibility

Bridge Section engineers develop planning cost estimates. The Chief Bridge Engineer reviews all planning estimates before they are sent to the regions.

8.1.9. Basis for Estimate

Base the planning estimates on historical cost data, location, the anticipated structure and foundation type, and estimated square foot of deck area.

Deck area is a function of the anticipated bridge length and width. Develop an approximate length from any existing site information or by increasing the existing bridge (if any) length by 10 percent then rounding up to a logical value.

Consult with the Statewide Hydraulics Engineer to verify the existing waterway opening prior to finalizing the preliminary bridge length.

For bridge width, match the proposed roadway width plus the width of the proposed bridge rails/barriers. Widths are sometimes determined by reviewing other recent projects in the same highway vicinity of the project.

Use Table 8-1 as a starting point for estimating the bridge construction costs. For more complicated or difficult projects, the bridge engineer should use the higher end of the range.

8.1.10. Ancillary Costs

The values in Table 8-1 incorporate only the basic bid item costs for typical bridges. Increase these values to reflect the ancillary costs to determine the estimated total project cost.

Either the Bridge Section or the region will determine these costs. In either case, clearly identify in the estimate whether the ancillary costs are included. Use the following guidance for adding contingency, mobilization, and construction engineering costs to the planning cost estimate:

Bridge Approach Roadway Costs

Include the approach roadway costs (sometimes called “logical touchdown costs”) in planning estimates if not included separately by the regions. Use 30 percent of the cost of the bridge for the cost of transitioning from the new bridge to the existing roadway. In the Department’s experience, this provides a conservative estimate suitable for planning and preliminary budgeting purposes.

Contingencies

For anticipated but undetermined costs, add a 20 to 30 percent contingency factor based on the sum of the estimated construction costs. The contingency factor is related to the amount of uncertainty in the hydraulic, foundation, and roadway geometric information at the time the estimate is being developed.

Mobilization and Demobilization

Add 10 percent of the basic bid item costs for the contractor’s mobilization and demobilization. This is the cost incurred by the contractor to mobilize the labor and equipment necessary for construction. A higher percentage may be justified for projects in remote areas.

Engineering

Add 15 to 20 percent of the estimated cost for preliminary engineering. Add 15 percent for construction engineering.

Indirect Cost Allocation Program (ICAP)

Add the percentage specified in the most recent Department ICAP memorandum to the total preliminary engineering and construction costs to account for Department overhead cost to construct the project.

For smaller projects, percentages for contingencies, mobilization, and engineering may be higher. For larger projects, these percentages may be lower.

Table 8-1
2023 Structural Material Cost Ranges*

	Structure Types	Typical Span Range (feet)	Estimated Cost
Hydraulic Structures	Corrugated Pipe	1-7	$250-$500 / LF
	Concrete Box Culvert	5-20	$500 -$4000 / LF
	Plate Arch Culvert	10-20	$250-$750 / LF
Structures for Conventional Site Conditions	Post-Tensioned Concrete Box Girder	120-240	$300-$600 / SF
	Prestressed Concrete Voided Slab	20-60	$350-$750 / SF
	Prestressed Concrete Decked Bulb-Tee	50-145	$300-$500 / SF
	Prestressed Concrete Girder	50-140	$400-$600 / SF
	Steel Rolled Girder	20-120	$400-$600 / SF
	Steel Rolled Girder with Precast Deck Panels	20-120	$400-$750 / SF
	Steel Plate Girder	60-400	$400-$750 / SF
	Steel Box Girder	100-400	$400-$800 / SF
	Timber	10-20	$250-$500 / SF
	Glulam Timber	15-40	$250-$500 / SF
	Plate Arch Railroad Tube	25-35	$2000-$5000 / SF
Structures for Special Site Conditions	Segmental Post-Tensioned Box Girder	200-600	$750-$1500 / SF
	Cable Stayed Bridge	600-1200	$1000-$2000 / SF
	Suspension Bridge	600-5000	$1000-$3000 / SF
	Arch Bridge	50-400	$1000-$2000 / SF
	Movable Span Bridge	200-350	> $2500 / SF
	Tunnel	30+	> $3000 / SF

* *While these cost ranges are reasonably accurate at the time of publication, they should be used with caution due to inflation and volatility in construction materials, equipment operation, and labor costs.*

8.2. Bridge Type Selection (Level 2)

At the Bridge Type Selection stage, the project manager submits a "start package" including preliminary highway alignment data, proposed typical section, and site contours including right-of-way limits. Lay out the bridge using the preliminary roadway plan, profile, typical section, site topography, and other available design information included in the start package. The layout should include estimated piles sizes, cast-in-place (CIP) concrete member sizes, span lengths, and riprap geometry. Most member sizes can be accurately estimated from past projects, geometric compatibility, and simplified structural analysis.

The project manager may request estimates for multiple bridge types during the development of the environmental document.

Select the most feasible, constructable, and usually the most economical type of structure to fit the project and develop General Layout and Site Plan sheets. This structure may differ from the structure envisioned in earlier estimates. Use the General Layout and Site Plan to calculate preliminary quantities and develop the Bridge Type Selection estimate.

8.2.8. Quantity Calculations

General Guidelines

The quantity calculations require rapid (typically, a day per bridge or less) but close approximation of the final bridge quantities. Include all items required for bridge construction, and use graphs and tables, similar bridges, and computations based on dimensions from the preliminary plans. The objective is to complete a relatively accurate estimate in a minimal amount of time. Bridge quantities are typically placed in a table on the General Layout or Site Plan Sheet.

Consider the following guidelines when calculating quantities:

1. **Pay Items**. Each pay item has an official title and item number that is tied to the *Standard Specifications for Highway Construction*, which are also listed in the Department's bid item database. The Department uses these coded item numbers for tracking and as a historic database. Cross check all items against the *Standard Specifications*, Standard Modifications, and Special Provisions to ensure the use of the appropriate pay items, methods of measurement, and basis of payment. Division 500 presents structural items. For some specialty or new items, the pay item number may not be in the database. If unable to locate a pay item, the bridge engineer may request a new pay item through the Regional Bid Tab Coordinator.
2. **Units of Measurement**. Record the quantity of all contract bid items consistent with the terms and units of measurement presented in the applicable item sections of the *Alaska Standard Specifications for Highway Construction.* Table 8-2 presents the units of measurements for typical bridge items.
3. **Computations**. The bridge engineer may use manual and computer methods to compute bridge quantities. Prepare computation sheets for each bid item and retain all computation sheets in the project file.
4. **Significant Digits**. In all quantity calculations, retain enough significant digits so that accuracy is neither sacrificed nor exaggerated. Calculate quantities for individual structure elements to one significant figure beyond the rounding value shown in Table 8-2, when applicable.
5. **Rounding**. Match exactly the quantity of any item provided in quantity summaries with the number provided on computation sheets. Round the total item quantity for a structure to the value shown in Table 8-2. Note any required rounding of a raw estimate on the computation sheet consistent with Table 8-2. Do not round the result of a quantity calculation until the value is ready for incorporation into the Bridge Basis of Estimate Table. Do not round lump-sum items and items measured as "each."
6. **Bridge Basis of Estimate Table**. Segregate the quantities into separate columns with respect to the substructure and superstructure. For lump-sum items, provide the calculated quantity for the item.

Lump-Sum Items

In general, DOT&PF uses the lump-sum method of payment for items of work that are easily defined and unlikely to vary. Lump-sum items are estimated using the units identified in Table 8-2. When a lump-sum item is developed without using an estimating unit (Removal of Structures and Obstructions, Cofferdams, etc.), note the basis of the estimated cost and any special circumstances or relevant information in the quantity calculation package. Ensure this information is properly addressed in the plans and Special Provisions.

Excavation and Riprap

Where possible at the Bridge Type Selection stage, round excavation and riprap quantities to the nearest 10 cubic yards. Do not provide the dimensions of width, length, and depth more accurately than the nearest foot.

Concrete

Calculate concrete subtotals for various elements (e.g., abutments, piers) in cubic feet to the nearest tenth of a cubic foot. Carry dimensions to the hundredth of a foot, except for a thin cross section multiplied by a large length (e.g., slab cross section) where rounding to one hundredth could produce a large discrepancy in the final quantity. After totaling all subtotals, convert the total to cubic yards and round according to Table 8-2.

8.2.9. Cost Estimate

The bridge engineer updates the structural costs used for the planning construction cost estimate at the Bridge Type Selection stage. The bridge engineer should make a reasonable estimate of the structure quantities (within ± 10 percent). If the quantities can be estimated, use the following procedure for the updated cost estimate:

Unit Costs

Review historic data from similar projects to determine the unit costs. Possible sources include bid tabulations maintained by the Bridge Section and *Means Heavy Construction Cost Data*, an industry publication. For elements with little or no known information, contact industry sources for possible unit cost guidance.

See Table 8-2.

Adjustments

Adjust the estimated construction cost to reflect the conditions at the bridge site. Adjustment factors may include:

- Geographic location
- Age of most recent construction costs
- Recent trends in cost of materials (e.g., shortages)
- Extent of falsework required
- Anticipated difficulty of construction
- Size of project relative to size of previous projects for which cost data is available
- Known foundation problems at the bridge site
- Anticipated construction logistics (e.g., traffic control during construction)
- Any other factors that are considered appropriate for the structure
- Judgment and experience of the bridge engineer

Inflation

Do not adjust the cost for expected inflation at the time of construction. However, note the date the cost estimate was prepared.

Ancillary Costs

The discussion on ancillary costs found in Section 8.1 applies to the Bridge Type Selection cost estimate, except as follows:

- Reduce the percentage of anticipated but undetermined costs as more quantities are calculated.
- Add 15 percent of the estimated cost for construction engineering.

Preliminary engineering costs rarely vary enough to justify a separate line in a Bridge Type Selection estimate. In the unusual case where preliminary engineering costs are significantly different between alternatives, include separate line items for preliminary engineering and construction engineering.

Life-Cycle Costs Analysis

When required by the Chief Bridge Engineer, conduct a life-cycle cost analysis for each design alternative. This analysis allows the user to compare different design lives for each alternative. TRB's *National Cooperative Highway Research Program (NCHRP) Report 483: Bridge Life-Cycle Cost Analysis* contains a methodology for bridge life-cycle cost analysis (BLCCA), software that automates the methodology, and a guidance manual for implementing BLCCA.

Table 8-2
Quantity Calculations and Cost Estimates
(Type Selection)
Page 1 of 5

Pay Item Number	Pay Item Name	Quantity	Method of Calculating Quantities	Rounding Accuracy	Unit Cost
202.0023.0000	Removal of Bridge	SF	Total area of existing bridge deck	1	$50/SF to $85/SF
205.0001.0000	Excavation for Structures	CY	Only needed when excavation exceeds that associated with "normal" work. In such cases, assume that the total volume is equal to the cross sectional area at centerline roadway times the estimated length perpendicular to the cross section.	10	$45/CY to $75/CY
205.0006.0000	Structural Fill	CY	Fill between the wingwalls times the maximum height from the bottom of the footing to the bottom of the approach slab or roadway section times 50 feet behind each backwall or end diaphragm.	1	$45/CY to $100/CY
501.0001.0000	Class A Concrete	CY	Volume of concrete based upon preliminary dimensions of columns/piles, cap beams, wingwalls, diaphragms, etc. Assume all pipe piles are filled with CIP concrete from the bottom of the cap down 55 feet.	0.1	$1000/CY to $2500/CY
501.0002.0000	Class A-A Concrete	CY	Volume of concrete based upon preliminary dimensions of CIP deck slabs and approach slabs.	0.1	$1500/CY to $2500/CY
501.0007.0000	Precast Concrete Members, Girders	LBS	Determine the number and cross sectional area of prestressed girders (typically 6 to 8 girders in the cross section for standard two-lane bridges) using the Department's bulb-tee program or other reasonable approach. Calculate the weight of each girder using 160 PCF.	1	$0.65/LB to $0.80/LB
503.0001.0000	Reinforcing Steel	LBS	Assume 4% of the total weight of all CIP concrete (Class A + Class A-A) for standard decked bulb-tee girder bridge. Assume 8% when drilled shafts are used.	10	$2.00/LB to $2.75/LB

Table 8-2
Quantity Calculations and Cost Estimates
(Type Selection)
Page 2 of 5

Pay Item Number	Pay Item Name	Quantity	Method of Calculating Quantities	Rounding Accuracy	Unit Cost
503.000 2.0000	Epoxy-Coated Reinforcing Steel	LBS	Assume 2% to 3% of the total weight of all CIP concrete (Class A + Class A-A) for standard decked bulb-tee girder bridge. Assume 6% when CIP decks are used.	10	\$2.75/LB to \$3.50/LB
504.000 1.0000	Structural Steel	LBS	Calculate the preliminary girder sized based upon standard depth-to-span ratio and slenderness ratios. In lieu of structural analysis, assume the following: Continuous spans: $w = 20+0.2*L$ Simple spans: $w = 25+0.3*L$ Where L is the maximum individual span length (feet) and w is the total unit weight of structural steel (pounds per square foot) including girders and all steel included in the pay item.	10	\$4.00/LB to \$6.50/LB \$10.00/LB for small quantities (e.g., seismic retrofit work)
505.000 5	Furnish Structural Steel Piles	LF	ABUTMENTS: For non-liquefiable soils, assume one HP 14 × 117 pile per girder reaction. For liquefiable soils, assume that one 24-inch diameter × ½ inch pipe pile is used per girder reaction. PIERS: For pier heights less than 15 feet, assume 36-inch diameter × ¾ inch pipe piles are used at a spacing of 10 feet. Otherwise, assume 48-inch diameter × 1-inch pipe piles are used at a spacing of 12 feet.	1	\$125/LF to \$175/LF for HP14X117 \$300/LF to \$400/LF for 36-inch diameter \$500/LF to \$750/LF for 48-inch diameter
505.000 6	Drive Structural Steel Piles	EA	The number of each type of pile to be driven	1	\$10,000/pile to \$15,000/pile for HP14x117 \$20,000/pile for 24-inch diameter \$30,000/pile to \$40,000/pile for 36-inch diameter \$40,000/pile to \$50,000/pile for 48-inch diameter

Table 8-2
Quantity Calculations and Cost Estimates
(Type Selection)
Page 3 of 5

Pay Item Number	Pay Item Name	Estimating Quantity	Method of Calculating Quantities	Rounding Accuracy	Unit Cost
507.000 1.0002	Steel Bridge Railing, 2-Tube	LF	Calculate the total two-tube bridge length including rail on the wingwalls from preliminary bridge layout	1	$250/LF to $300/LF
507.000 1.0003	Steel Bridge Railing, 3-Tube	LF	Calculate the total three-tube bridge length including rail on the wingwalls from preliminary bridge layout	1	$250/LF to $350/LF
507.000 4.0000	Concrete Barrier	LF	Calculate the total bridge barrier length including rail on the wingwalls from preliminary bridge layout	1	$250/LF to $325/LF
508.000 1.0000	Waterproofing Membrane	SF	Calculate total membrane area on bridge deck and approach slabs between curbs.	1	$10/SF to $15/SF
511.000 1.0000	Mechanically Stabilized Earth Wall	SF	Calculate the total area of wall face based upon preliminary bridge layout. Area of wall is measured from top of coping to bottom of wall panels.	1	$50/SF to $75/SF
512.000 2.0000	Falsework	SF	Typically the cost of falsework is included in the 501 pay items – that is, it is subsidiary to CIP concrete. Falsework may be bid as a separate pay item for CIP box girders or other non-standard bridge types are specified that require significant amounts of falsework. If falsework is bid as a separate pay item, reduce the unit cost of CIP concrete.	1	$50/SF to $100/SF
513.000 1.0000	Field Painting of Steel Structures	SF	Avg. value of 125 SF per ton of structural steel. Table 3 on page 270 of "Good Painting Practice Third Edition," Steel Structures Painting Council provides more detail. Calculate actual surface area and check conversion factors from this table on large, deep plate and box girders.	1	$35/SF to $50/SF for (E) bridges w/ zinc paint. $40/SF to $75/SF for (E) bridges w/ lead paint
513.000 2.0000	Field Metallizing of Steel Structures	SF	Calculate using the same methods as 513.0001.0000		$40/SF to $60/SF

Table 8-2
Quantity Calculations and Cost Estimates
(Type Selection)
Page 4 of 5

Pay Item Number	Pay Item Name	Estimating Quantity	Method of Calculating Quantities	Rounding Accuracy	Unit Cost
520.0001 .0000	Temporary Crossing	SF	Calculate the deck area based upon the estimated length and width required for the detour bridge. Typical bridge width is 34 feet out-to-out.	1	$125/SF to $175/SF
606.0016 .0000	Transition Rail	EA	Typically, there is one thrie beam transition railing at each end of the bridge railing – 4 per bridge.	1	$3000/EA to $4000/EA
611.0001 .0002	Riprap, Class II	CY	Calculate the volume of riprap using the preliminary bridge layout. The riprap volume can be estimated as the cross section area times the length of the riprap measured along the center-of-gravity of the cross section. In lieu of calculations, use 1000 CY for "normal" riprap usage and 2500 CY for "extensive" riprap usage.	10	$75/CY to $125/CY In remote locations, $250/CY depending upon local availability
631.0002 .0001	Geotextile, Erosion Control, Class 1	SY	Calculate the area of geotextile from the preliminary bridge layout. In lieu of calculations, use the same area of geotextile in SY as volume of riprap in CY.	10	$3.00/SY to $6.00/SY
640.0001 .0000	Mobilization and Demobilization	%	The mobilization and demobilization is typically taken to be 10% of the total construction cost. Thus, use 11.1% of the subtotal bridge bid cost.		Subtotal of all other bridge bid prices time 1/9
NA	Construction Engineering	%	Use 15% of the total estimated construction cost		Total bridge cost including mobilization times 15%
NA	ICAP	%	This "overhead" cost changes every year. Use the published value. In lieu of the published value, use 5%.		Total bridge plus construction engineering cost times ICAP %

Table 8-2
Quantity Calculations and Cost Estimates
(Type Selection)
Page 5 of 5

Pay Item Number	Pay Item Name	Estimating Quantity	Method of Calculating Quantities	Rounding Accuracy	Unit Cost
	Contingency	%	For preliminary designs, include a contingency that is reflective of the available information. Use the following: 1. 15% to 20% for 1R and 3R projects depending on the complexity and number of bridges 2. 20% to 25% when roadway, hydraulic and foundation information is available and used in the preliminary bridge layout 3. 25% to 30% when roadway information is available and an existing bridge is being replaced but no specific hydraulic or foundation data is available 4. 30% to 35% when roadway information is available but no hydraulic or foundation information is available and the proposed bridge is not reasonably close to any existing bridge		Total bridge cost including ICAP times the appropriate factor

8.3. PS&E Stage (Level 3)

8.3.8. Quantity Calculations

Update Calculations

The bridge engineer must revisit all quantity calculations as determined at the Bridge Type Selection stage (see Section 8.2.1) and update as necessary.

Reinforcing Steel

The reinforcing steel quantity is based on the weight of the reinforcing bar. Calculate bar lengths to hundredths of a foot. Accumulate total lengths of bar in each size before weights are extended to reduce rounding errors. Use Table 8-3 to obtain the weight in pounds for the appropriate reinforcing steel size. Do not include the weight of incidental items in this quantity.

Structural Steel

The structural steel quantity is based on the weight of the steel components in the structure. The contract documents present structural steel as a lump sum bid item, but also provide the steel weight as information only to assist the contractor in preparing the bid. Include the weight of all beams, plates, diaphragms, stiffeners, bearing plates, bolts and nuts, shear studs, rockers, rollers, pins and nuts, expansion dams, roadway drains and scuppers, weld metal and structural shapes for expansion joints, and pier protection in the estimate.

Calculate the weight of the steel using the following guidelines:

1. **Lengths.** Carry lengths to the hundredth of a foot.
2. **Unit Weights.** Structural steel has a weight density of 490 lb/feet^3. See the *Alaska Standard Specifications* for the weight density of other common metals.
3. **Shapes, Plates, Railing, and Flooring.** Calculate the nominal weights and dimensions as shown on the contract drawings, deducting for copes, cuts, and open holes, exclusive of bolt holes.
4. **Casting.** Compute the weights for castings from the dimensions shown on the plans with a 5 percent allowance for fillets and overruns. Deduct the weight for drillings or borings. The bridge engineer may use scale weights for castings of small complex parts, because it could be difficult to compute their weight accurately.
5. **Bolts, Nuts, and Washers.** Measure bolts, nuts, and washers for payment based on the computed weight as presented in the *Alaska Standard Specifications.*
6. **Fillet Welds.** Estimate fillet welds using Table 8-4 and the *Alaska Standard Specifications*.
7. **Rounding.** Round the total of the structural steel to the nearest 10 lbs.

8.3.9. Cost Estimate

Update Estimate

The bridge engineer must revise the cost estimate from the Bridge Type Selection stage (see Section 8.2.2) to reflect the final bid items and quantities. No "contingency" is used at the PS&E stage.

Submit the final estimate to the project manager in spreadsheet format along with the plans and Special Provisions. Some project managers also want a copy of the quantity calculations, and these should be submitted with the PS&E package if requested.

The project manager merges the cost estimates for structural and other project pay items to determine the Engineer's Estimate for the project.

Engineer's Estimate

The Engineer's Estimate is the final estimate used for programming construction funding. It lists the total quantity and estimated price for each pay item. Pay item quantities in the Engineer's Estimate must match the estimate of quantities in the plans and the bid schedule in the bidding documents. The project manager uses the Engineer's Estimate as a basis for requesting Authority to Advertise (ATA) a project. After bids are received, the Engineer's Estimate provides a basis for determining the reasonableness of the bids.

Design Aid Formulas

The following figures present various mathematical relationships to assist the bridge engineer with quantity estimating:

- Figure 8-1 — Slope Equations
- Figure 8-2 — Trigonometric Solution of Triangles
- Figure 8-3 — Area of Plane Figures
- Figure 8-4 — Surface Area and Volume of Solids

Table 8-3
Reinforcing Steel Sizes and Weights

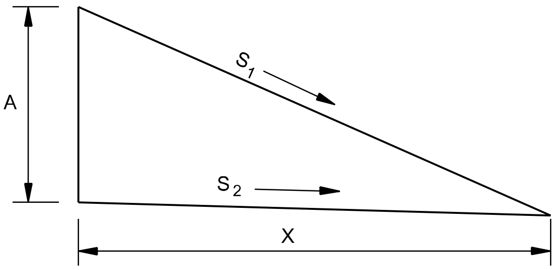
Bar Size		Nominal Properties		
		Diameter	Area	Weight
US Customary Designation	Metric Designation	(in)	(in^2)	(lb/ft)
#3	#10	0.375	0.11	0.376
#4	#13	0.500	0.20	0.668
#5	#16	0.625	0.31	1.043
#6	#19	0.750	0.44	1.502
#7	#22	0.875	0.60	2.044
#8	#25	1.000	0.79	2.670
#9	#29	1.128	1.00	3.400
#10	#32	1.270	1.27	4.303
#11	#36	1.410	1.56	5.313
#14	#43	1.693	2.25	7.650
#18	#57	2.257	4.00	13.600

Table 8-4
Weights of Fillet Welds

Nominal Size (in)	1/4	5/16	3/8	1/2	5/8	3/4	7/8	1
Weight (lb/ft)	0.20	0.25	0.35	0.55	0.80	1.10	1.50	2.00

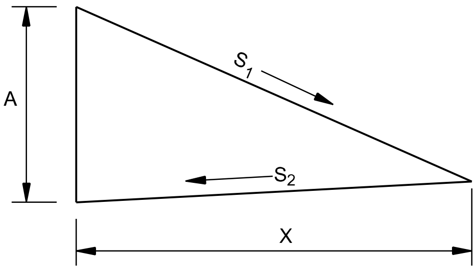
Note: Estimate other welds based on their theoretical dimension plus 50% for overrun.

CASE I



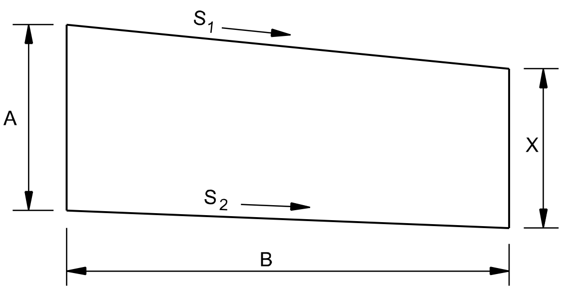
$$X = \frac{A}{S_1 - S_2}$$

CASE II



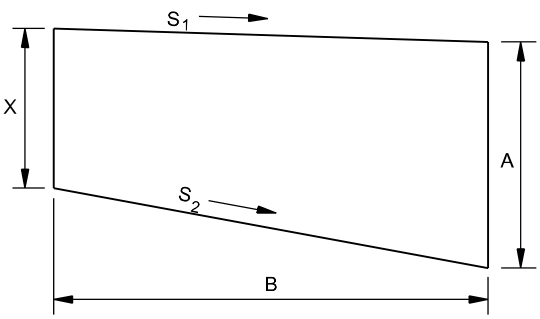
$$X = \frac{A}{S_1 + S_2}$$

CASE III



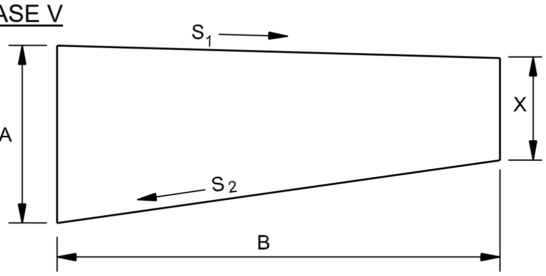
$X = A + (S_2 - S_1)\,B$ Area $= \frac{A + X}{2}\,(B)$

CASE IV



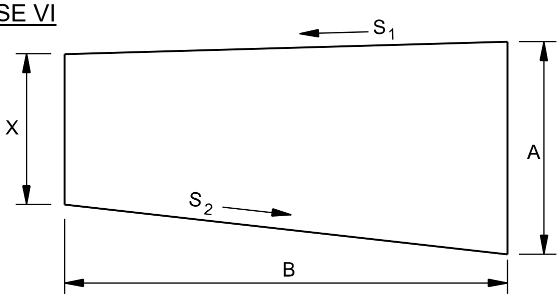
$X = A + (S_2 - S_1)\,B$ Area $= \frac{A + X}{2}\,(B)$

CASE V



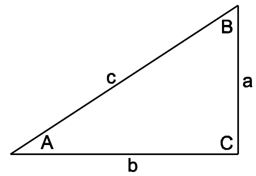
$X = A - (S_1 + S_2)\,B$ Area $= \frac{A + X}{2}\,(B)$

CASE VI



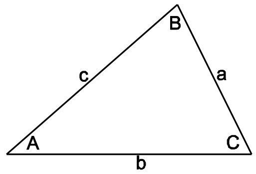
$X = A + (S_1 + S_2)\,B$ Area $= \frac{A + X}{2}\,(B)$

Figure 8-1
Slope Equations



Right-Angled Triangles		
Given:	**Sought:**	**Formulae:**
a,c	A,B,b	$\sin A = \frac{a}{c}$, $\cos B = \frac{a}{c}$, $b = \sqrt{c^2 - a^2}$
	Area	Area = $\frac{a}{2}\sqrt{c^2 - a^2}$
a,b	A,B,c	$\tan A = \frac{a}{b}$, $\tan B = \frac{b}{a}$, $c = \sqrt{a^2 + b^2}$
	Area	Area = $\frac{ab}{2}$
A,a	B,b,c	$B = 90° - A$, $b = a \cot A$, $c = \frac{a}{\sin A}$
	Area	Area = $\frac{a^2 \cot A}{2}$
A,b	B,a,c	$B = 90° - A$, $a = b \tan A$, $c = \frac{b}{\cos A}$
	Area	Area = $\frac{b^2 \tan A}{2}$
A,c	B,a,b	$B = 90° - A$, $a = c \sin A$, $b = c \cos A$
	Area	Area = $\frac{c^2 \sin A \cos A}{2}$ or $\frac{c^2 \sin 2A}{4}$

Figure 8-2
Trigonometric Solution of Triangles
Page 1 of 2



$$s = \frac{a+b+c}{2}$$

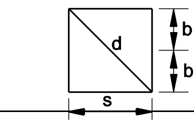
Oblique-Angled Triangles

Given:	Sought:	Formulae:
a,b,c	A	$\sin\frac{1}{2}A = \sqrt{\frac{(s-b)(s-c)}{bc}}$, $\cos\frac{1}{2}A = \sqrt{\frac{s(s-a)}{bc}}$, $\tan\frac{1}{2}A = \sqrt{\frac{(s-b)(s-c)}{s(s-a)}}$
	B	$\sin\frac{1}{2}B = \sqrt{\frac{(s-a)(s-c)}{ac}}$, $\cos\frac{1}{2}B = \sqrt{\frac{s(s-b)}{ac}}$, $\tan\frac{1}{2}B = \sqrt{\frac{(s-a)(s-c)}{s(s-b)}}$
	C	$\sin\frac{1}{2}C = \sqrt{\frac{(s-a)(s-b)}{ab}}$, $\cos\frac{1}{2}C = \sqrt{\frac{s(s-c)}{ab}}$, $\tan\frac{1}{2}C = \sqrt{\frac{(s-a)(s-b)}{s(s-c)}}$
	Area	$\text{Area} = \sqrt{s(s-a)(s-b)(s-c)}$
a,A,B	b,c	$b = \frac{a\sin B}{\sin A}$ $c = \frac{a\sin C}{\sin A} = \frac{a\sin(A+B)}{\sin A}$
	Area	$\text{Area} = \frac{1}{2}ab\sin C = \frac{a^2\sin B\sin C}{2\sin A}$
a,b,A	B	$\sin B = \frac{b\sin A}{a}$
	c	$c = \frac{a\sin C}{\sin A} = \frac{b\sin C}{\sin B} = \sqrt{a^2+b^2-2(ab)(\cos C)}$
	Area	$\text{Area} = \frac{1}{2}(ab)(\sin C)$
$(2bc)(\cos A)$ a,b,C	A	$\tan A = \frac{a\sin C}{b-a\cos C}$ $\tan\frac{1}{2}(A-B) = \frac{a-b}{a+b}\cot\frac{1}{2}C$
	c	$c = \sqrt{a^2+b^2-2ab\cos C} = \frac{a\sin C}{\sin A}$
	Area	$\text{Area} = \frac{1}{2}(ab)(\sin C)$

$a^2 = b^2 + c^2 - (2bc)(\cos A)$, $b^2 = a^2 + c^2 - (2ac)(\cos B)$, $c^2 = a^2 + b^2 - (2ab)(\cos C)$

Figure 8-2
Trigonometric Solution of Triangles
Page 2 of 2

Square



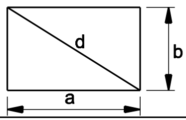
Diagonal = $d = s\sqrt{2}$

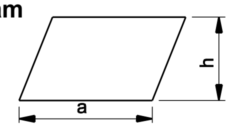
Area = $s^2 = 4b^2 = 0.5\ d^2$

Example: $s = 6$; $b = 3$; Area = $(6)^2 = 36$

$d = 6 \times 1.414 = 8.484$

Rectangle and Parallelogram



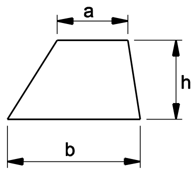


Area = ab or $b\sqrt{d^2 - b^2}$

Example: $a = 6$; $b = 3$

Area = $3 \times 6 = 18$

Trapezoid

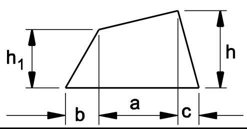


Area = $\frac{1}{2}h\left(a+b\right)$

Example: $a = 2$; $b = 4$; $h = 3$

Area = $\frac{1}{2} \times 3\left(2+4\right) = 9$

Trapezium



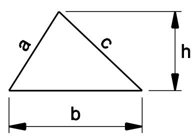
Area = $\frac{1}{2}\left[\ a\left(h+h_1\right) + bh_1 + ch\ \right]$

Example: $a = 4$; $b = 2$; $c = 2$; $h = 3$; $h_1 = 2$

Area = $\frac{1}{2}\left[4\left(3+2\right)+\left(2\times 2\right)+\left(2\times 3\right)\right] = 15$

Triangles

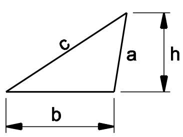
Formulas apply to both figures.



Area = $\frac{1}{2}\ bh$

Example: $h = 3$; $b = 5$

Area = $\frac{1}{2}\left(3\times 5\right) = 7.5$

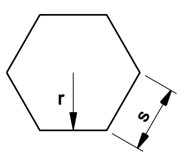


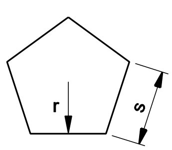
Area = $\sqrt{s\left(s-a\right)\left(s-b\right)\left(s-c\right)}$ where: $s = \frac{a+b+c}{2}$

Example: $a = 2$; $b = 3$; $c = 4$

$s = \frac{2+3+4}{2} = 4.5$; Area = $\sqrt{4.5\left(4.5-2\right)\left(4.5-3\right)\left(4.5-4\right)} = 2.9$

Regular Polygons





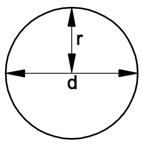
$$
\text{Area}\begin{cases}
5 \text{ sides} = 1.720477\ s^2 = 3.63271\ r^2 \\
6 \text{ sides} = 2.598150\ s^2 = 3.46410\ r^2 \\
7 \text{ sides} = 3.633875\ s^2 = 3.37101\ r^2 \\
8 \text{ sides} = 4.828427\ s^2 = 3.31368\ r^2 \\
9 \text{ sides} = 6.181875\ s^2 = 3.27573\ r^2 \\
10 \text{ sides} = 7.694250\ s^2 = 3.24920\ r^2 \\
11 \text{ sides} = 9.365675\ s^2 = 3.22993\ r^2 \\
12 \text{ sides} = 11.196300\ s^2 = 3.21539\ r^2
\end{cases}
$$

n = number of sides; r = short radius; s = length of side; R = long radius.

Area = $\frac{n}{4}s^2\cot\frac{180°}{n} = \frac{n}{2}R^2\sin\frac{360°}{n} = nr^2\tan\frac{180°}{n}$

Figure 8-3
Area of Plane Figures
Page 1 of 3

Circle



$\pi = 3.1416$; A = area; d = diameter

p = circumference or periphery; r = radius

$p = \pi d = 3.1416d$ $\qquad p = 2\sqrt{\pi A} = 3.54\sqrt{A}$

$p = 2\pi r = 6.2832r$ $\qquad p = \frac{2A}{r} = \frac{4A}{d}$

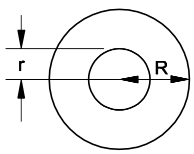
$d = \frac{p}{\pi} = \frac{p}{3.1416}$ $\qquad d = 2\sqrt{\frac{A}{\pi}} = 1.128\sqrt{A}$

$r = \frac{p}{2\pi} = \frac{p}{6.2832}$ $\qquad r = \sqrt{\frac{A}{\pi}} = 0.564\sqrt{A}$

$A = \frac{\pi d^2}{4} = 0.7854d^2$ $\qquad A = \frac{p^2}{4\pi} = \frac{p^2}{12.57}$

$A = \pi r^2 = 3.1416r^2$ $\qquad A = \frac{pr}{2} = \frac{pd}{4}$

Circular Ring



Area = $\pi\left(R^2 - r^2\right) = 3.1416\left(R^2 - r^2\right)$

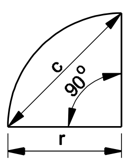
Area = $0.7854\,(D^2 - d^2) = 0.7854(D - d)(D + d)$

Area = difference in areas between the inner and outer circles.

Example: R = 4; r = 2

Area = $3.1416(4^2 - 2^2) = 37.6992$

Quadrant

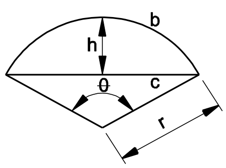


Area = $\frac{\pi r^2}{4} = 0.7854\,r^2 = 0.3927c^2$

Example: r = 3; c = chord

Area = $0.7851 \times 3^2 = 7.0686$

Segment



b = length of arc; θ = angle in degrees; c = chord = $\sqrt{4\left(2hr - h^2\right)}$

Area = $\frac{1}{2}\left[br - c(r - h)\right] = \pi r^2 \frac{\theta}{360} - \frac{c(r - h)}{2} p$

When θ is greater than 180°, then $\frac{c}{2}$ x difference between r and h is added

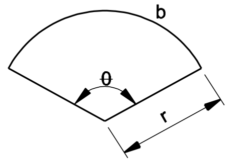
to the fraction $\frac{\pi r^2 \theta}{360}$

Example: r = 3; θ = 120°; h = 1.5

Area = $3.1416 \times 3^2 \times \frac{120}{360} - \frac{5.196\,(3 - 1.5)}{2} = 5.5278$

Figure 8-3
Area of Plane Figures
Page 2 of 3

Sector



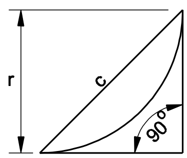
Area = $\frac{br}{2} = \pi r^2 \frac{\theta}{360°}$

θ = angle in degrees; b = length of arc

Example: r = 3; θ = 120°

Area = $3.1416 \times 3^2 \times \frac{120}{360} = 9.4248$

Spandrel

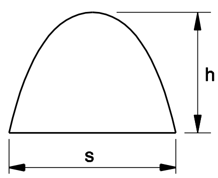


Area = $0.2146\ r^2 = 0.1073\ c^2$

Example: r = 3

Area = $0.2146 \times 3^2 = 1.9314$

Parabola



l = length of curved line = periphery – s

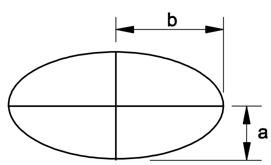
$l = \frac{s^2}{8h}\left[\sqrt{c(1+c)} + 2.0326 \times \log\left(\sqrt{c} + \sqrt{1+c}\right)\right]$ where $c = \left(\frac{4h}{s}\right)^2$

Area = $\frac{2}{3}sh$

Example: s = 3; h = 4

Area = $\frac{2}{3} \times 3 \times 4 = 8$

Ellipse



Area = $\pi ab = 3.1416ab$

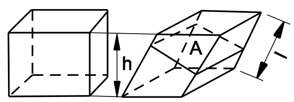
Circumference = $2\pi\sqrt{\frac{a^2 + b^2}{2}}$ (close approximation)

Example: a = 3; b = 4.

Area = 3.1416 x 3 x 4 = 37.6992

Circumference = $2 \times 3.1416\sqrt{\frac{(3)^2 + (4)^2}{2}} = 6.2832 \times 3.5355 = 22.21$

Figure 8-3
Area of Plane Figures
Page 3 of 3

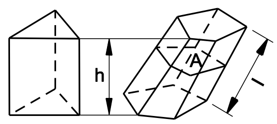


Parallelopiped

S = perimeter (P) perpendicular to sides x lateral length (l): Pl

V = area of base (B) x perpendicular height (h): Bh

V = area of section (A) perpendicular to sides x lateral length (l): Al

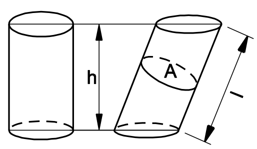


Prism, Right, or Oblique, Regular or Irregular

S = perimeter (P) perpendicular to sides x lateral length (l): Pl

V = area of base (B) x perpendicular height (h): Bh

V = area of section (A) perpendicular to sides x lateral length (l): Al



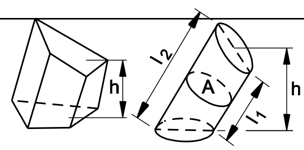
Cylinder, Right, or Oblique, Circular or Elliptic, etc.

S = perimeter of base (P) x perpendicular height (h): Ph

S = perimeter (P_1) perpendicular to sides x lateral length (l): P_1l

V = area of base (B) x perpendicular height (h): Bh

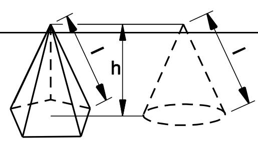
V = area of section (A) perpendicular to sides x lateral length (l): Al



Frustum of any Prism or Cylinder

V = area of base (B) x perpendicular distance (h) from base to center of gravity of opposite face: Bh

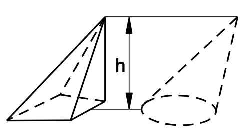
For cylinder: $\frac{1}{2} A \left(l_1 + l_2\right)$



Pyramid or Cone, Right and Regular

S = perimeter of base (P) x $\frac{1}{2}$ slant height (l): $\frac{1}{2}Pl$

V = area of base (B) x $\frac{1}{8}$ perpendicular height (h): $\frac{1}{8}Bh$

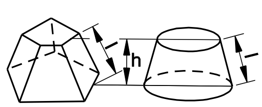


Pyramid or Cone, Right, or Oblique, Regular or Irregular

V = area of base (B) x $\frac{1}{3}$ perpendicular height (h): $\frac{1}{3}Bh$

V = $\frac{1}{3}$ volume of prism or cylinder of same base and perpendicular height

V = $\frac{1}{2}$ volume of hemisphere of same base and perpendicular height



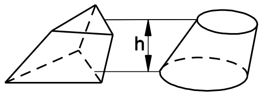
Frustum of Pyramid or Cone, Right and Regular, Parallel Ends

S = (sum of perimeter of base (P) and top (p)) x $\frac{1}{2}$ slant height (l): $\frac{1}{2} l \, (P + p)$

V = (sum of areas of base (B) and top (b) + square root of their products) x $\frac{1}{3}$ perpendicular height (h): $\frac{1}{3} h \left(B + b + \sqrt{Bb}\right)$

S = Lateral or Convex Surface Area *V = Volume*

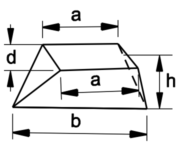
Figure 8-4
Surface Area and Volume of Solids
Page 1 of 3



Frustum of any Pyramid or Cone, Parallel Ends

V = (sum of areas of base (B) and top (b) + square root of their products)

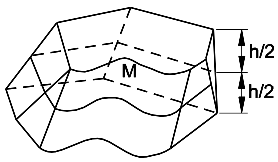
x $\frac{1}{8}$ perpendicular height (h): $\frac{1}{8}\,h\left(B + b + \sqrt{Bb}\right)$



Wedge, Parallelogram Face

V = $\frac{1}{6}$ (sum of three edges (a, b, a) x perpendicular height (h)

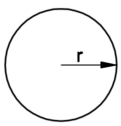
x perpendicular width (d): $\frac{1}{6}\,d\,h\,(2a+b)$



Prismatoid

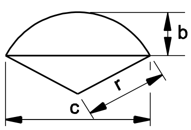
V = $\frac{1}{6}$ perpendicular height (h) x (sum of areas of base (B) and top (b) + 4 x area of section (M) parallel to bases and midway between them): $\frac{1}{6}\,h(B + b + 4M)$

The Prismatoid formula applies also to any of the foregoing solids with parallel bases, to pyramids, cones and spherical sections, and to many solids with irregular surfaces.



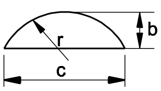
Sphere

$S = 4\pi r^2 = \pi d^2 = 3.14159265\,d^2$ $\quad$ $V = \frac{4}{3}\pi r^3 = \frac{1}{6}\pi d^3 = 0.52359878\,d^3$



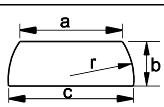
Spherical Sector

$S = \frac{1}{2}\pi r(4b + c)$ $\quad$ $V = \frac{2}{3}\pi r^3 b$



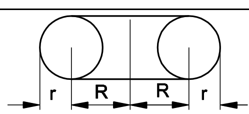
Spherical Segment

$S = 2\pi r b = \frac{1}{4}\pi\left(4b^2 + c^2\right)$ $\quad$ $V = \frac{1}{3}\pi b^2(3r - b) = \frac{1}{24}\pi b\left(3c^2 + 4b^2\right)$



Spherical Zone

$S = 2\pi\, r b$ $\quad$ $V = \frac{1}{24}\pi b\left(3a^3 + 3c^2 + 4b^2\right)$



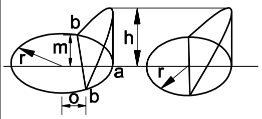
Circular Ring

$S = 4\pi^2 Rr$ $\quad$ $V = 2\pi^3 Rr^2$

S = Lateral or Convex Surface Area *V = Volume*

Figure 8-4
Surface Area and Volume of Solids
Page 2 of 3

Ungula of Right, Regular Cylinder



Base = Segment, b a b

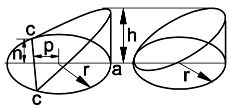
$$S = (2\,r\,m - o \times \text{arc, } b\,a\,b)\ \frac{h}{r-o}$$

$$V = \left(\frac{2}{3}m^3 - o \times \text{area, } b\,a\,b\right)\frac{h}{r-o}$$

Base = Half Circle

$$S = 2rh$$

$$V = \frac{2}{3}r^2\,h$$



Base = Segment, c a c

$$S = (2rn + p \times \text{arc, } c\,a\,c)\ \frac{h}{r+p}$$

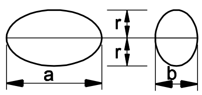
$$V = \left(\frac{2}{3}n^3 + p \times \text{area, } cac\right)\frac{h}{r+p}$$

Base = Circle

$$S = r\,\pi\,h$$

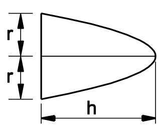
$$V = \frac{1}{2}r^2\,\pi\,h$$

Ellipsoid



$$V = \frac{1}{3}\pi\,r\,a\,b$$

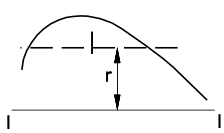
Paraboloid



$$V = \frac{1}{2}\pi\,r^2 h$$

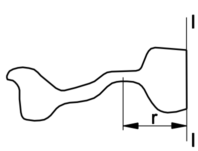
Ratio of corresponding volumes of a Cone, Paraboloid, Sphere, and Cylinder of equal height: $\frac{1}{3} : \frac{1}{2} : \frac{2}{3} : 1$

Bodies Generated by Partial or Complete Revolution



$\left.\begin{array}{l} \text{l = length of a curve} \\ \text{A = area of a plane} \end{array}\right\}$ rotating about an axis l - l on one side and in plane of axis

r = distance of center of gravity of line or plane from axis l - l and for any angle of revolution, a°



$\frac{2\,r\,\pi\,a°}{360}$ = length of arc described by center of gravity

S = length of curve x length of arc about axis = $l\ \frac{2\,r\,\pi\,a°}{360}$

For complete revolution, $S = 2\,r\,\pi\,l$

V = area of plane x length of arc about axis = $A\ \frac{2\,r\,\pi\,a°}{360}$

For complete revolution, $V = 2\,r\,\pi\,A$

S = Lateral or Convex Surface Area *V = Volume*

Figure 8-4
Surface Area and Volume of Solids
Page 3 of 3

9. Design Quality Control (QC) Procedures

9.1 General

9.1. General

This section describes the requirements for the designer, checker, and engineer of record (EOR) in reviewing and finalizing the PS&E package.

The bridge portion of the PS&E is ready for review and approval once the Plans reach 95 percent completion. The following defines the process for completing and signing the Final Bridge Plans:

1. **Estimate**. The bridge designer is responsible for estimating the quantities and cost of the bridge bid items.
2. **Plan Review**. The bridge checker will:
   - prepare independent calculations to verify the bridge plan details are structurally adequate and meet the standards of this *Manual*.
   - verify plan sheets are correct and complete ("biddable & buildable"). The checker will mark every item on every plan sheet as either “OK” or “No Good.” Highlighting or color coding is an acceptable means of marking the plans.
   - verify the Engineer's Estimate is correct and complete by calculating quantities and reviewing item prices for reasonableness
   - verify that all bid items are covered by the *Standard Specifications for Highway Construction* or a Special Provision
3. **Revisions/Corrections**. The marked up plans will be returned to the bridge designer for consideration, correction, or clarification. The designer and checker will meet to resolve all discrepancies, and the designer will revise the plans as needed. Upon completing the revisions, the designer will submit the bridge plans to the Bridge Squad Leader for quality control review.
4. **Quality Review**. The Squad Leader or Chief Bridge Engineer will conduct a quality review of the bridge plans. The plans are reviewed to determine if they meet the following requirements:
   - the approved project scope of work and Design Study Report
   - the design criteria presented in the *Alaska Bridges and Structures Manual*, except where revised by a design variance
   - the criteria presented in Bridge Design Memorandums
   - any reports or studies for the project
   - the plan preparation information presented in this *Manual*
   - good bridge engineering and detailing practice
5. **Plans Submittal Process for Bid Package**. After the plans are reviewed, the EOR (who is either the designer or checker) will sign and submit the plans, quantity and cost estimates, and special provisions to the Chief Bridge Engineer. Once the Chief Bridge Engineer is satisfied with the final plans, the bridge plans are submitted to the Project Manager for incorporation into the overall contract documents. The Project Manager submits the final plans, special provisions, and quantity and cost estimates to the Regional Contracts Section.

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10. Introduction to Part II

10.1. Overview

Features of the *Alaska Bridges and Structures Manual* (the Manual) include:

Application. The *Manual* is an application-oriented document.

Theory. The *Manual* is not a structural design theory resource or a research document, but does provide background information for DOT&PF's bridge design criteria and application.

Example Problems. The *Manual* provides a few example problems or calculations demonstrating the proper procedure for selected bridge design applications. These design examples or calculations illustrate the specific structural design criteria, practices, and policies used by DOT&PF for the indicated applications. However, example problems do not absolve the bridge designer from the responsibility of understanding their applicability.

Details. Where beneficial, the *Manual* provides details for the various structural design elements.

Coordination with AASHTO *LRFD Bridge Design Specifications*. Part II of the *Manual* is a supplement to the AASHTO *LRFD Bridge Design Specifications* (*LRFD Specifications*), that:

- in general, does not duplicate information in the *LRFD Specifications*, unless necessary for clarity
- elaborates on specific articles of the *LRFD Specifications*
- presents interpretative information
- indicates DOT&PF's policy where the *LRFD Specifications* presents multiple options; and
- presents bridge design applications used in Alaska that are not included in the *LRFD Specifications*

In addition, the *Manual* discusses, for selected applications, the original development of the *LRFD Specifications* to assist the bridge engineer.

10.2. Manual Application

In general, the *Manual* applies to all bridge design projects under the responsibility of DOT&PF. These projects include DOT&PF projects, local agency projects using federal funds, state-funded projects, and possibly other projects that will use the *Alaska Standard Specifications*.

10.2.1. Audience

The primary audience for the *Manual* is the bridge engineer in the DOT&PF Bridge Section. In addition, the *Manual*'s audience includes:

- other DOT&PF personnel
- consultants retained by DOT&PF
- contractors retained by DOT&PF for design/build projects
- local agencies where the project is funded with federal and/or state money

10.2.2. Order of Precedence

The provisions cited in 23 CFR 625.4(b) are minimum standards that must be followed in all cases. If the *Alaska Bridge and Structures Manual* exceeds these standards, the *Manual* will govern. In the event of conflict between AASHTO publications, the more conservative requirements or interpretation will govern. The Chief Bridge Engineer will make any final determination in the case of unresolved conflicts.

10.2.3. Scope of Work Definitions

The appropriate application of the structural design criteria in this *Manual* will depend, in part, on the scope of the proposed structural work.

New Bridge: This scope of work is defined as a new bridge at a new location. With rare exceptions, the bridge engineer will meet the criteria presented in this *Manual* for all new bridge projects.

Bridge Replacement: This scope of work is where the existing bridge requires complete replacement of the superstructure, substructure, and foundation due to structural or functional deficiencies of the existing structure. The replaced bridge will generally maintain the horizontal and vertical alignment of the existing approaching roadway. With rare exceptions, the bridge engineer will meet the criteria presented in this *Manual* for the structural design of bridge replacement projects.

Bridge Widening: It may be necessary to widen an existing bridge for a variety of reasons where the existing superstructure and substructure are considered structurally sound. Types of bridge widenings include:

- widening an existing bridge without adding travel lanes
- adding travel lanes to a highway segment to increase the traffic-carrying capacity of the facility
- adding an auxiliary lane across the structure (e.g., adding a truck-climbing lane; and
- adding a bike path or pedestrian facility

See Chapter 23 for the application of the *Manual* to bridge widening projects.

Major Bridge Rehabilitation: Any number of deficiencies may indicate the need for major bridge rehabilitation:

- deterioration of structural elements
- insufficient load-carrying capacity
- inadequate seismic resistance
- traffic safety features (e.g., substandard bridge rail, substandard guardrail-to-bridge-rail transition); and/or
- geometric deficiencies (e.g., narrow bridge width, inadequate horizontal alignment)

Major bridge rehabilitation is warranted where it is more cost-effective than replacement. For DOT&PF practices on bridge rehabilitation, see Chapter 23.

Bridge Deck Rehabilitation/Replacement: If the bridge substructure and foundation are structurally sound, but the bridge deck is deficient, the deck may require rehabilitation or replacement. This decision will depend upon the level and type of rehabilitation required, cost-effective analysis, traffic volumes, and safety issues.

Bridge deck rehabilitation may also consist of one or more of the elements listed below under "Minor Bridge Rehabilitation." See Chapter 23 for DOT&PF practices on bridge deck rehabilitation.

Minor Bridge Rehabilitation: Minor bridge rehabilitation is generally limited to deficiencies with isolated bridge elements that exceed the limits of routine maintenance. Minor bridge rehabilitation work may include the following types of activities:

- expansion joint cleaning, repair, or replacement
- deck patching and/or sealing
- deck protection
- spot painting of structural steel; and/or
- drains and drainage systems

Seismic Retrofit: Highway bridges are vulnerable to partial or total collapse during earthquakes due to three main reasons:

- girders dropping from their supports
- seismic moments and shears exceeding the capacity of the columns, and
- ground failure associated with liquefaction and lateral spread

Seismic retrofit attempts to improve the performance of existing bridges that are vulnerable to these failure modes. Retrofits typically fall into one of two categories:

1. **Phase I**. The primary objective of Phase I retrofit strategies is to prevent girders from falling off their supports (abutment and pier caps) in a relatively cost-effective manner. The girders can be tied to their supports with restrainer cables to limit their displacement relative to the supports. Seismic retrofit can also limit girder movement by installing concrete shear keys. On some bridges, timber blocking can be installed between the ends of the girders and the abutment backwall to limit longitudinal girder movement. Another strategy involves increasing the support width, thus increasing the displacement capacity.
2. **Phase II**. All retrofit work beyond Phase I is Phase II retrofit strategies. Phase II strategies attempt to improve substructure ductility, and they are generally more expensive than Phase I strategies. They typically address column seismic deficiencies through retrofits such as column jacketing and foundation deficiencies through footing modifications. Seismic base isolation is generally categorized as a Phase II retrofit.

Bridge Maintenance: This scope of work consists of activities considered routine maintenance (e.g., repairing damaged bridge rail, cleaning out drainage inlets).

10.3. Design Variances

This section discusses DOT&PF procedures for identifying, justifying, and processing variances to the structural design criteria in the *Manual* and *LRFD Specifications*. For bridges in remote sites, see Chapter 20.

10.3.1. DOT&PF Intent

Meet all design criteria in this *Manual* and the *LRFD Specifications* including current interims. The effective application date of interim provisions is when the AASHTO Subcommittee on Bridges and Structures adopts them or as soon as practicable afterwards. Recognizing that strict adherence to design standards may not always be practical, DOT&PF has established a process to evaluate and approve variances to its structural design criteria.

10.3.2. Procedures

Formal, written approvals for variances are required where the criteria or policies in either this *Manual* or the *LRFD Specifications* are presented in one of the following contexts (or the like):

- "shall"
- "mandatory"
- "required" or
- if the statement is written in directive language to the bridge engineer (e.g., "use")

When proposing a design element that does not meet the requirements of the *Manual* or *LRFD Specifications* in the above context, follow the procedure outlined below.

Documentation

Prepare the justification for the variance at the earliest possible stage of the project. Address the following issues, as applicable:

- site constraints
- safety considerations
- construction costs
- construction logistics
- product availability
- environmental impacts, or
- right-of-way impacts

Document any proposed variances from the DOT&PF structural design criteria in the Bridge Type Selection Report (see Chapter 4). At a minimum, document any design variances in the design computations after the Bridge Type Selection Report is approved. Document significant changes in a memo and include it in the bridge file.

Approval

The Chief Bridge Engineer, in accordance with the current FHWA Stewardship and Oversight Agreement, must approve all proposed variances in writing. Figure 10-1 presents a typical memorandum used to seek design variance approval.

Forward a copy of the memorandum to the Engineering Manager for the Preconstruction Engineer's approval according to Chapter 11 of the *Alaska Highway Preconstruction Manual*.

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES

M E M O R A N D U M

(date) , 2

To: *(name)*, Chief Bridge Engineer

From: *(name)*, Bridge Engineer

Subject: Design Variance Request
Project No. *(project number)*

Attached for your review and approval for the subject project is a request for a design variance to Section *number and title* of the *Alaska Bridges and Structures Manual* and/or Article *number and title* of the AASHTO *LRFD Bridge Design Specifications*. The Attachment is pursuant to Section 10.3 "Design Variances" of the *Alaska Bridges and Structures Manual*.

Approved: *(name)* , Chief Bridge Engineer

Attachment

Figure 10-1
Design Variance Request

10.4. Structural Design Literature (National)

This section discusses the major national publications available in the structural design literature. It provides 1) a brief discussion on each publication, and 2) the status and application of the publication by DOT&PF. This list is not all inclusive; however, it does represent a hierarchy of importance. In all cases, bridge engineers must ensure that they are using the latest edition of the publication, including all interim revisions to date unless otherwise directed in this *Manual*.

10.4.1. LRFD Bridge Design Specifications

Description

The AASHTO *LRFD Bridge Design Specifications* serve as the national standard for use by bridge engineers or for the development of a transportation agency's own structural specifications. The *LRFD Specifications* establish minimum requirements, consistent with current nationwide practices that apply to common highway bridges and other structures such as retaining walls and culverts. Long-span structures may require design provisions in addition to those presented in the *LRFD Specifications*.

Interim revisions are issued annually. AASHTO currently intends to publish a completely updated edition every two years.

LRFD Methodology

The *LRFD Specifications* present a load-and-resistance-factor design (LRFD) methodology for the structural design of bridges, which replaces the load factor design (LFD) and service load design (SLD) methodologies of the previous AASHTO *Standard Specifications for Highway Bridges* (*Standard Specifications*).

The *LRFD Specifications* apply live-load factors that are lower than the traditional AASHTO load factors but balance this reduction with an increase in vehicular live load that more accurately models actual loads on our nation's highways. Basically, the LRFD methodology requires that bridge components be designed to satisfy four sets of limit states:

- Strength
- Service
- Fatigue-and-Fracture
- Extreme Event

Through the use of statistical analyses, the provisions of the *LRFD Specifications* reflect a uniform level of safety for all structural elements, components, and systems.

Superseded Publications

The information in the *LRFD Specifications* supersedes, partially or completely, several AASHTO structural design publications. As such, AASHTO no longer maintains these publications. However, some of these publications contain background information or other presentations that may be useful.

- *Standard Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections*. This publication provides information on the inelastic design of compact steel members (resistance beyond first yield), historically known as autostress.

  An Appendix to the *LRFD Specifications* contains an updated inelastic design process for compact steel sections.

- *Guide Specifications for Strength Design of Truss Bridges*. This document provides provisions for the design of steel trusses using the Load Factor Design (LFD) methodology. Herein, the load combination for long-span bridges (i.e., the Strength IV load combination of the *LRFD Specifications*) first appeared.

- *Division 1A, Seismic Design of the Standard Specifications for Highway Bridges*. See Section 10.4.3.

- *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*. This publication provides recommended requirements for identifying, fabricating, welding, and testing fracture critical, non-redundant steel bridge members. It includes specifications on welding requirements that are in addition to those in the ANSI/AASHTO/AWS *Bridge Welding Code*.

  This document also discusses the need for proper identification of fracture critical members on plans, and it contains useful information addressing background, example problems, etc., that are not included in the *LRFD Specifications*.

- *Guide Specifications — Thermal Effects in Concrete Bridge Superstructures*. This publication provides guidance on the thermal effects in concrete superstructures with special attention to the thermal gradient through the depth of the superstructure. These provisions have been incorporated into the *LRFD Specifications*.

- *Guide Specifications for Fatigue Design of Steel Bridges*. This publication provides an alternative procedure to that of the AASHTO *Standard Specifications for Highway Bridges* wherein the actual number of cycles are used for fatigue design. Such a procedure has now been adopted in the *LRFD Specifications*.
- *Guide Specifications for Design and Construction of Segmental Concrete Bridges*. This document provides details on the design and construction of segmental concrete bridges. The high points have subsequently been included in the *LRFD Bridge Design Specifications* and the *LRFD Bridge Construction Specifications*.
- *Guide Specifications for Structural Design of Sound Barriers*. This document provides criteria for the structural design of sound barriers to promote the uniform preparation of plans and specifications. The publication allows the design of masonry sound barriers in addition to concrete, wood, steel, synthetics, and composites and aluminum.
- *Guide Specifications for Horizontally Curved Highway Bridges*. The AASHTO *Guide Specifications for Horizontally Curved Highway Bridges* presents specifications and methodologies for the design of steel I-girder and steel box girder bridges that are on a horizontal curve. The design methodology is based on both the service load and load factor design methodologies and, therefore, is not compatible with the *LRFD Specifications*. DOT&PF only allows the use of this publication for the same applications as for the *Standard Specifications*.

DOT&PF Application

DOT&PF has adopted the use of the *LRFD Specifications* as augmented by this *Manual* as the mandatory document for the structural design of highway bridges and other structures. The *Manual* is based upon the current edition of the *LRFD Specifications* with current interim revisions. This policy does not apply to:

- existing elements for bridge widening and bridge rehabilitation projects (including seismic retrofits), originally designed to any edition of the *AASHTO Standard Specifications for Highway Bridges*, where strengthening is not involved. These modifications may be designed to the *AASHTO Standard Specifications for Highway Bridges*, 17th Edition, AASHTO, 2002.
- structural elements for which no LRFD specifications are available

Part II of the *Manual* presents DOT&PF's specific applications of the *LRFD Specifications* to structural design, which clarify, specify among options, or augment information from the *LRFD Specifications* for DOT&PF's application.

10.4.2. Guide Specifications for LRFD Seismic Bridge Design

These *Guide Specifications* are an alternative set of provisions for the seismic design of highway bridges. The major difference between these provisions and those in the *LRFD Specifications* is the methodology used for examining seismic demands. Because the methodology of the *Guide Specifications* focuses on displacements, it is often referred to as "displacement-based." By contrast, the seismic provisions in the *LRFD Specifications* are "force-based."

DOT&PF Application

Use the *Guide Specifications for LRFD Seismic Bridge Design* for the seismic design of new highway bridges, including temporary bridges.

10.4.3. Standard Specifications for Highway Bridges

The AASHTO *Standard Specifications for Highway Bridges* (*Standard Specifications*) was first published in the late 1920s with annual interim revisions and, until the adoption of the *LRFD Specifications*, served as the national standard for the design of highway bridges. The final version of the *Standard Specifications* is based on the Service Load Design (SLD) and Load Factor Design (LFD) methodologies. AASHTO maintained the AASHTO *Standard Specifications* through 2000, and published the final comprehensive 17th Edition in 2002.

DOT&PF Application

See Chapter 23 for the use of the *Standard Specifications* on bridge widening and rehabilitation projects. Where the *Standard Specifications* apply, HS20 is the minimum highway live load for strength considerations in the application of the *Standard Specifications*. Use HS25 live-load on all Interstate bridges, major hauling routes, routes accessing major shipping points (including the Port of Anchorage), and access routes to identified resource areas. The HS25 live-load model is defined as 1.25 times the HS20 live loading as provided in the *Standard Specifications*.

10.4.4. Seismic Retrofitting Manual for Highway Bridges, May 1995*

The FHWA *Retrofitting Manual* is based primarily on research conducted during the development of the 1983 FHWA guidelines by the Applied Technology Council, current Caltrans Seismic Design Criteria, and recent research conducted at the University of California, San Diego and elsewhere. The *Retrofitting Manual* offers procedures for evaluating and upgrading the seismic resistance of existing highway bridges. Specifically it contains:

- a preliminary screening process to identify and prioritize bridges that need to be evaluated for seismic retrofitting;
- a methodology for quantitatively evaluating the seismic capacity of an existing bridge and determining the overall effectiveness of alternative seismic retrofitting measures; and
- retrofit measures and design requirements for increasing the seismic resistance of existing bridges.

The *Retrofitting Manual* does not prescribe requirements dictating when and how bridges require a retrofit. The decision to retrofit a bridge depends on a number of factors, several of which the *Retrofitting Manual* does not address. These include, but are not limited to, the availability of funding and political, social, and economic considerations.

* ***Note:*** *This document was replaced by the FHWA Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges, December 2009; however, Alaska's program will continue to be based on the 1995 Retrofitting Manual.*

DOT&PF Application

Use the *Seismic Retrofitting Manual for Highway Bridges*, May 1995, for retrofitting of highway bridges.

10.4.5. Guide Specifications for Seismic Isolation Design

AASHTO published the *Guide Specifications for Seismic Isolation Design* as a supplement to the *Standard Specifications for Seismic Design of Highway Bridges*. The *Guide Specifications for Seismic Isolation Design* presents specifications for the design of bearings to seismically isolate the superstructure from the substructure of highway bridges.

DOT&PF Application

Use this guide, where applicable, in conjunction with the *Guide Specifications for LRFD Seismic Bridge Design*.

10.4.6. ANSI/AASHTO/AWS Bridge Welding Code D1.5M/D1.5

The *Bridge Welding Code* presents current criteria for the welding of structural steel in bridges. The *Bridge Welding Code* superseded the AASHTO *Standard Specifications for Welding of Structural Steel Highway Bridges* and supplements the *Structural Welding Code*, AWS D1.1.

The *Code* includes a commentary on selected sections.

DOT&PF Application

DOT&PF has adopted the mandatory use of the *Bridge Welding Code D1.5* for the design and construction of structural steel highway bridges. However, for items not specifically addressed in D1.5, such as welding on existing structures, welding tubular members, or welding on reinforcing steel, refer to the current edition of ANSI/AWS D1.1 and ANSI/AWS D1.4.

10.4.7. LRFD Guide Specifications for Design of Pedestrian Bridges

The AASHTO *LRFD Guide Specifications for Design of Pedestrian Bridges* apply to bridges intended to carry primarily pedestrian traffic, bicycle traffic, or both. This document is based upon the LRFD design methodology.

DOT&PF Application

Use this guide for the design of pedestrian bridges in conjunction with the *LRFD Specifications*.

10.4.8. Guide Specifications for Distribution of Loads for Highway Bridges

The AASHTO *Guide Specifications for Distribution of Loads for Highway Bridges* provides more refined live-load distribution factors than the traditional S-over factors of the *Standard Specifications*. Although the refined equations appear similar, they are not the same as those provided in the *LRFD Specifications* and shall not be used with the *LRFD Specifications*. The equations in the *Guide Specifications* include the multiple-presence factors of the *Standard*

Specifications, while those of the *LRFD Specifications* include the differing multiple-presence factors inherent to it.

DOT&PF Application

This publication only applies to non-LRFD applications. Therefore, this document is only used when reverting back to the *Standard Specifications* for design.

10.4.9. Guide Design Specifications for Bridge Temporary Works

The AASHTO *Guide Design Specifications for Bridge Temporary Works* is used in construction specifications for falsework, formwork, and related temporary construction used to build highway bridge structures.

DOT&PF Application

Reference this publication in construction contracts where applicable. Note that this publication is not the appropriate standard for temporary detour bridges.

10.4.10. LRFD Movable Highway Bridge Design Specifications

The AASHTO *LRFD Movable Highway Bridge Design Specifications* addresses the design of movable highway bridges (including ferry transfer bridges) using the *LRFD Bridge Design Specifications*. This document provides guidance for the structural design and machinery design of swing, bascule and vertical-lift spans.

DOT&PF Application

Use this publication for the design of any movable bridges.

10.4.11. LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

The AASHTO *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* present structural design criteria for the supports of various roadside appurtenances. The publication presents specific criteria and methodologies for evaluating dead load, live load, ice load, and wind load. This publication also includes criteria for several types of materials used for structural supports such as steel, aluminum, concrete, and wood.

DOT&PF Application

Use this publication when designing signs, luminaires, and signals.

10.4.12. Guide Specification and Commentary for Vessel Collision Design of Highway Bridges

The *AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges* is a comprehensive publication that includes information relative to designing bridges to resist vessel collision damage. As feasible, it is based on probabilistic principles. The *LRFD Specifications* contain only the load section of this document. The *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges* contains more comprehensive information.

DOT&PF Application

Use this guide for the design of vulnerable highway bridges.

10.4.13. AISC Steel Construction Manual

The *Steel Construction Manual*, published by the American Institute of Steel Construction (AISC), provides dimensions, properties, and general design guidance for structural steel for various applications. The AISC *Manual* contains AISC allowable stress design and load-and-resistance factor design method criteria for steel buildings. However, the properties of the rolled structural shapes are sometimes useful for designing bridge structures.

DOT&PF Application

Use this AISC manual only where it addresses items not in the *LRFD Specifications* and with the approval of the Chief Bridge Engineer.

10.4.14. AREMA Manual for Railway Engineering

The AREMA *Manual for Railway Engineering*, published by the American Railway Engineering and Maintenance-of-Way Association (AREMA), provides detailed structural specifications for the design of railroad bridges. The AREMA specifications have approximately the same status for railroad bridges as the *LRFD Specifications* have for highway bridges; i.e., the structural design of railroad bridges must meet the AREMA requirements.

DOT&PF Application

Occasionally, DOT&PF is responsible for the structural design of railroad bridges. Use the specifications of the AREMA *Manual*, except as modified by the Alaska Railroad Corporation. In addition, the AREMA *Manual* contains requirements for the geometric design of railroad tracks passing beneath a highway bridge. See Chapter 24.

10.4.15. Other Structural Design Publications

Many of the other available structural design publications may, on a case-by-case basis, be useful. The design engineer is responsible for verifying the adequacy of any member proportions, details or practices in these publications to ensure that they are consistent with the *LRFD Specifications*. The following briefly describes several of these structural design publications:

- *Prestressed/Precast Concrete Institute (PCI) Design Handbook*. This publication includes information on the analysis and design of precast and/or prestressed concrete products in addition to a discussion on handling, connections, and tolerances for prestressed products. It contains general design information, specifications, and standard practices.
- *Prestressed/Precast Concrete Institute (PCI) Bridge Design Manual*. This design manual includes both preliminary and final design information for standard girders and precast, prestressed concrete products and systems used for transportation structures. This document contains background, strategies for economy, fabrication techniques, evaluation of loads, load tables, design theory, and numerous complete design examples. This publication explains and amplifies the application of both the *Standard Specifications* and *LRFD Specifications*.
- *Post-Tensioning Institute (PTI) Post-Tensioning Manual*. This publication discusses the application of post-tensioning to many types of concrete structures, including concrete bridges. It also discusses types of post-tensioning systems, specifications, the analysis and design of post-tensioned structures, and their construction.
- *Concrete Reinforcing Steel Institute (CRSI) Handbook*. This publication meets the ACI Building Code Requirements for Reinforced Concrete.
- *National Steel Bridge Alliance (NSBA) Highway Structures Design Handbook*. This document addresses many aspects of structural steel materials, fabrication, economy, and design, and it includes LRFD examples; the general computational procedure is helpful to bridge engineers using the *LRFD Specifications*.
- *American Concrete Institute (ACI) — Analysis and Design of Reinforced Concrete Bridge Structures*. This publication contains information on various concrete bridge types, loads, load factors, service and ultimate load design, prestressed concrete, substructure and superstructure elements, precast concrete, and reinforcing details.
- *CRSI Manual of Standard Practice*. This publication explains generally accepted industry practices for estimating, detailing, fabricating, and placing reinforcing bars and bar supports.
- *PTI — Post-Tensioned Box Girder Bridges.* This publication contains information on economics, design parameters, analysis and detailing, installation, prestressing steel specifications, post-tensioning tendons, systems, and sources.
- *United States Department of Agriculture (USDA) Forest Service Timber Bridge Manual*. This *Manual* addresses all aspects of traditional timber bridge construction plus the latest developments in laminated deck systems using adhesives or prestressing forces.
- *Timber Construction Manual*. This document, published by the American Institute of Timber Construction (AITC), provides criteria for the design of timber structures, including bridges, for both sawn and laminated timber.
- *International Building Code*. This document, published by the International Conference of Building Officials (ICBO), provides criteria for the design of buildings.
- NCHRP 343 *Manuals for Design of Bridge Foundations*. This publication provides additional information on the application of the *LRFD Specifications* to foundations.
- *American Concrete Institute (ACI) 318-05 Building Code Requirements for Structural Concret*e. This document addresses the proper design and construction of structural concrete buildings. Although this document is intended for building design, bridge engineers find it useful because it provides more detail on aspects of concrete design that are less typical in highway bridges.

- PCA *Notes on ACI 318-02 Building Code Requirements for Structural Concrete with Design Applications*. The PCA Notes assist the engineer in the proper application of the ACI 318-02 design standard, which is the predecessor to ACI 318-05. Numerous design examples illustrate the application of the *Code* provisions.

10.5. *Alaska Highway Preconstruction Manual*

10.5.1. General

The *Manual* includes the following chapters:

- Chapter 2 “Organization”
- Chapter 4 “Project Development Process”
- Chapter 5 “Public Involvement and Agency Coordination”
- Chapter 10 “Highway Capacity”
- Chapter 11 “Highway Design”
- Chapter 12 “Non-Motorized Transportation”
- Chapter 14 “Traffic Safety in Work Zones”

Chapters 4 and 11 have the most application to the development of bridge plans by the Bridge Section.

10.5.2. Chapter 4 “Project Development Process”

Chapter 4 establishes procedures and guidance for developing federal-aid and state-funded highway capital improvement projects from project development authorization to construction contract award. The chapter addresses the following topics:

- preparing the environmental document
- preparing the Design Study Report (DSR)
- developing a set of project construction plans
- conducting field reviews
- advertising and award

10.5.3. Chapter 11 “Highway Design”

Chapter 11 of the *Alaska Highway Preconstruction Manual* is the preeminent design standard for the Department. This chapter interprets, amends, and augments AASHTO policies for application to DOT&PF projects. All designs should satisfy the minimum values in the *Manual* consistent with topographic, cultural, and economic conditions.

Several sections within Chapter 11 apply directly to bridges (e.g., vertical clearance, bridge rails). Review these sections when designing a bridge.

10.6. Bridge Identification

10.6.1. Structure Number Assignment

The Bridge Management Unit is responsible for assigning structure numbers to all structures. The four-digit structure numbers are assigned sequentially. 0001 to 2999 are reserved for highway bridges, 4001 to 4999 are used for highway culverts, 6001 to 6999 are DOT&PF-owned pedestrian minor structures, and 7001 to 7999 are DOT&PF-owned minor structures carrying highway traffic.

10.6.2. Bridge Type Names

Name bridge structures in accordance with the information provided below and in Figure 10-2. The following applies:

Bridges crossing rivers, creeks, streams, and other bodies of water.

The bridge is named for the body of water being crossed (e.g., Gastineau Channel Bridge). Adding the location (e.g., Tanana River Bridge at Tok) or the name of the road (e.g., Campbell Creek Bridge at C Street) can further modify this name.

Railroad grade separation structures

These are always either “overheads” or “underpasses.” Standing on the railroad and describing the relative location of the street or highway identifies the proper name. These structures are further named by their location or the street or highway involved. For example, the Eklutna Overhead carries the Glenn Highway *over* the Alaska Railroad near Eklutna. The Parks Highway passes *under* the railroad tracks at the Parks Highway Underpass.

Local streets and roads crossing state highways at “overcrossings” or “undercrossings”

Standing on the highway and describing the relative location of the street or road identifies the proper name. These structures are further named by the street involved (e.g., the Muldoon Overcrossing carries Muldoon Road *over* the Glenn Highway).

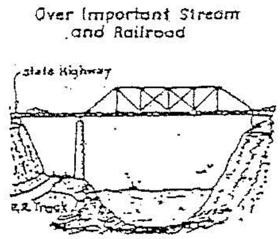
Two state highways cross each other at a “separation”

Alaska has few separation structures.

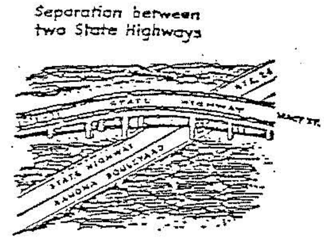
Pedestrian pathways

These use the same names as indicated above with the addition of the word “pedestrian.” For example, Montana Creek Pedestrian Bridge. Pedestrian overcrossings (POCs) are frequently named for their location or vicinity. For example, the East High POC is located adjacent to East High School in Anchorage.

BRIDGE

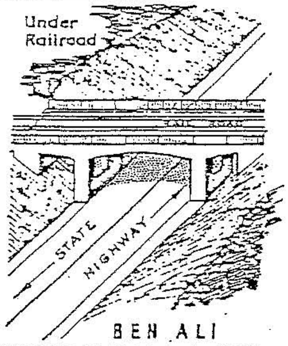


BRIDGE



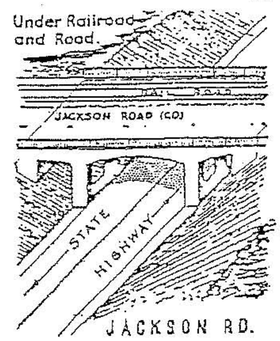
MACY-RAMONA
(OR RTE 2-RTE 26)

SEPARATION



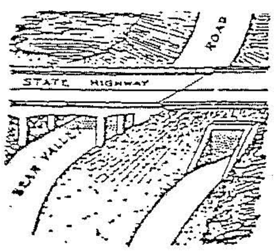
BEN ALI

UNDERPASS



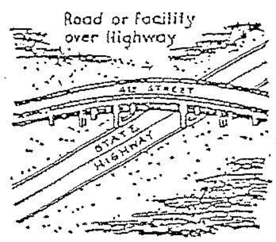
JACKSON RD.

UNDERPASS



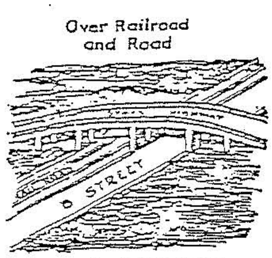
BEAR VALLEY RD &
BEAR VALLEY PEDESTRIAN

UNDERCROSSING



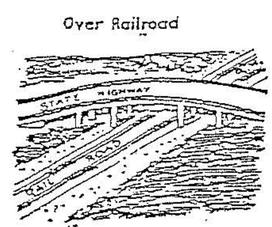
4TH STREET

OVERCROSSING



B STREET

OVERHEAD



BEN ALI

OVERHEAD

Figure 10-2
Bridge Type Names

11. Structural Systems and Dimensions

11.1. Introduction

Chapter 4 discusses the importance of bridge type selection and the content and format of the Bridge Type Selection Report. Chapter 11 discusses the type selection of the major components of the bridge (i.e., superstructure, substructure, foundation), and this chapter discusses other factors that impact these decisions (e.g., hydraulics, roadway design elements). In general, the chapter has been organized to present the decision-making process from the location of the bridge to the structure-type selection for the site.

11.1.1. Alaska-Specific Issues

The AASHTO *LRFD Bridge Design Specifications* are intended for general application nationwide. When compared to the contiguous United States, however, the state of Alaska presents challenges to bridge design, construction, and maintenance due to the extreme climate, rugged terrain, and a generally remote location. Therefore, some flexibility in the application of the *LRFD Specifications* is necessary to account for these challenges. Consider the issues outlined below when consulting this chapter.

1. **Construction:** The short construction season in Alaska, due to the cold climate, favors accelerated bridge construction (ABC) techniques. Further, DOT&PF prefers designs that do not require two or more construction seasons. Bridge construction sites can be very remote dictating a preference for pre-fabricated components of modest proportions.

2. **Availability of Construction Materials:** Depending upon the location within the state, contractors must transport a significant portion of the bridge materials long distances to the bridge site. Few or no local construction materials and fabricated bridge components may be available. In many cases, fabricated bridge components must be shipped from the contiguous US.

3. **Design:** Seasonal climate extremes can result in large design temperature ranges. These temperature ranges dictate a preference for certain bridge or component types.

4. **Maintenance:** Due to access issues for remote sites, DOT&PF prefers bridges requiring little or no maintenance. Life-cycle cost considerations justify higher initial costs, such as metalizing steel bridges to reduce maintenance costs.

5. **Environmental Considerations:** The environmental impacts from proposed highway and bridge projects in Alaska can be an especially sensitive issue and may involve unique design constraints.

6. **Construction Costs:** Ultimately, all of the preceding factors impact bridge construction costs.

   Initial costs for bridges in Alaska may be higher than in the contiguous US; however, life-cycle cost considerations suggest that Alaska bridges are not inordinately more expensive.

Because of the unique issues associated with remote construction sites, Chapter 20 specifically addresses bridge design and construction for these sites.

11.2. Bridge Location

Establishing the location of a bridge is an interactive process among DOT&PF units responsible for roadway design, bridge design, hydraulics, geotechnical engineering, right-of-way, and environment. The process of selecting a bridge location should consider economics, ease of construction, minimizing environmental impacts, and optimizing service for the traveling public.

The DOT&PF regional project managers are responsible for coordinating the activities of the various Department units in selecting the bridge location. Bridge location, in turn, will impact the structure-type selection. This section summarizes the significant factors that impact the decision.

11.2.1. Roadway Design

Roadway design factors that impact bridge location and structure-type selection include the following:

- horizontal alignment (e.g., tangent, curve, superelevation, skew)
- vertical clearances and alignment (e.g., longitudinal gradient, vertical curves)
- traffic volumes
- roadway width
- presence of medians, sidewalks, bike lanes
- clear zones through underpasses

The roadway designer establishes the roadway alignment. Ideally, bridges are located on a tangent alignment with no skew, width changes, or superelevation transitions. However, project constraints may not allow this.

Although bridges can be designed to accommodate almost any given geometry, the bridge engineer must work closely with the roadway designer to minimize the adverse effect of some of the following roadway design issues.

Horizontal Alignment

Where bridges are constructed on horizontal curves, the bridge design, geometry, and construction becomes complicated and the number of feasible bridge types is limited. The structural analysis of horizontally curved bridges with small radii of curvature requires a refined computer analysis.

Hand-calculation methods are available but are accurate only for horizontally curved bridges of large radii. Only use hand calculations as a check of the refined analysis. See Chapter 13 for guidance on acceptable analysis methods.

In general, structural steel and cast-in-place concrete are best suited for horizontally curved bridges.

Decked bulb-tee girders are suited for larger radius curves where the deck curvature can be accommodated by the top flange. Decked girders can also accommodate a limited amount of superelevation transition (e.g., 1 percent change in cross slope).

Skew

Skews of less than approximately 30 degrees are acceptable for most bridge types and result in moderate detailing challenges. Bridges with skews of more than 30 degrees require the Chief Bridge Engineer's approval.

Some structure types with skews more than 30 degrees may require a refined analysis. Analyze all structures with skews of more than 60 degrees by refined methods.

Bridges having a skew of more than 60 degrees may also have long-term functionality problems such as uplifting of girders in the acute corners, lateral translation of the bridge bearings, or both. Consider alternatives to these highly skewed bridges if practical. See Chapter 13 for guidance on acceptable methods for analyzing bridges of varying skew.

Vertical Alignment

Vertical alignment is not typically considered in the structural analysis of bridges; however, vertical alignment is reflected in the calculated camber for steel and precast concrete girders. Most bridges with significant vertical curvature require perpendicular placement of the deck-finishing machine.

All bridges require some elevation change in vertical alignment to allow for deck drainage. Use a minimum of a 0.5 percent longitudinal gradient or a crest vertical curve to accommodate bridge deck drainage.

11.2.2. Hydraulics

The hydraulics engineer will prepare a Hydraulics Report in accordance with the *Alaska Highway Drainage Manual* and provide preliminary hydraulic recommendations in coordination with the Bridge Section's structure-type selection. The critical hydraulic factors may include the following:

- channel geometry
- peak flow discharge
- design water surface elevation(s)
- skew angle
- freeboard requirements, including navigational openings
- scour and erosion potential
- flood plain management objectives
- water-related regulatory requirements

Locate bridges to accommodate natural channel processes and to minimize affects on channel morphology. Many rivers in Alaska are highly dynamic, with complex braid plains and channel networks, high rates of bank erosion and channel migration, and varying flow distributions between channels. Some are subject to glacial outburst floods, ice jams, aufeis accumulation, or other cold region challenges.

The hydraulics engineer and state foundation engineer provide preliminary information to determine the potential for scour at each proposed site.

Spur dikes and other forms of channel control are sometimes needed to ensure favorable flow alignments under bridges. The hydraulics engineer will provide guidance and recommendations regarding the need for these controls.

Responsibilities of Hydraulics Engineer

The hydraulics engineer is responsible for hydrologic and hydraulic analyses for bridge waterway openings. The hydraulics engineer performs the following for the design of bridge waterway openings for bridges:

- hydrologic analysis to determine the design flow rates
- hydraulic analysis to determine the channel geometry and protection provisions needed to meet hydraulic engineering objectives;
- scour analysis for foundation design
- river ice consideration to assess risks of ice jams, aufeis accumulation, or other ice-related processes
- deck drainage analysis to determine the need for deck drains or other stormwater controls. This task is usually performed in coordination with roadway designers, regional hydraulic engineers, bridge engineers, and environmental analysts.

Based on the hydraulic analysis, the hydraulics engineer will provide the following to the bridge engineer for new bridges:

1. water surface elevation(s) for the design peak flow discharge. DOT&PF bridge plans generally include the 2 percent, 1 percent, and 0.2 percent annual exceedence probability discharges.
2. freeboard requirements for ice or debris
3. range of anticipated ice thicknesses along the study reach
4. bridge waterway opening dimensions and skew angle
5. results of the hydraulic scour analysis
6. channel and abutment protection measures

The hydraulics engineer also determines whether the bridge design is consistent with environmental regulations related to hydraulics engineering, such as flood plains.

Hydraulic Definitions

The following presents selected hydraulic definitions that have an application to bridge design:

Annual Exceedence Probability (AEP) Discharge: The term preferred by the scientific community for describing discharge values based upon statistical analysis of streamflow gaging records of annual peaks. A 1 percent AEP discharge is synonymous to the “100-year flood.”

Aufeis: (German for “on ice”) A form of ice accumulation or sheet-like mass of layered ice that forms from successive flows of ground water over an ice surface during freezing temperatures. This form of ice is also commonly referred to as “icings.”

Backwater: The incremental increase in water surface elevation upstream of a highway facility usually associated with a channel encroachment.

Base Flood: The flood having a 1 percent chance of being exceeded in any given year (i.e., the 1 percent AEP discharge and “100-year flood”).

Base Floodplain: The area subject to flooding by the base flood.

Breakup: (of river ice). The downstream transport of ice along rivers and creeks during the seasonal thaw of spring. Spring breakup can also involve ice jams with severe backwater and flooding.

Bridge Waterway Opening: The opening provided in the roadway embankment intended to pass the stream flow under the design conditions.

Design Flood Frequency: The flood frequency selected for determining the necessary size of the bridge waterway opening.

Design Surface Water Elevation: The water elevation produced by the design flood.

Flood Frequency: The number of times a flood of a given magnitude can be expected to occur on average over a long period of time.

Freeboard: The clearance between the design water surface elevation and the low chord of the superstructure.

Ice Jam: A phenomenon of ice accumulation in a channel causing a disruption to the downstream movement of ice and water. Ice jams can cause severe backwater events.

Maximum Allowable Backwater: The maximum amount of backwater that is acceptable to DOT&PF for a proposed facility based on local ordinances, state and federal laws, and on DOT&PF policies.

Ordinary High Water (OHW): The mark along the bank up to which the presence and action of the non-tidal water are so common and usual, and so long continued in all ordinary years, as to leave a natural line impressed on the bank or shore and indicated by erosion, shelving, changes in soil characteristics, destruction of terrestrial vegetation, or other distinctive physical characteristics (see Alaska Statutes 41.17.950(15) for the legal definition). This line shall be located and established during the site topographic survey or location hydraulic survey. The OHW line is a legal boundary and should be surveyed by licensed professional surveyors if needed for boundary control purposes (e.g., property lines, delineations of jurisdictional authority).

Overtopping Flood: That flood event that will overtop the elevation of the bridge or roadway approaches.

Peak Discharge: (Also: Peak Flow or Peak Streamflow.) The maximum volumetric rate of water flow for a given time period. The peak discharge for the 1 percent AEP flood or "100-year flood" is expressed as Q_{100}.

Recurrence Interval: (Also: Return Period). This term refers to the estimated number of years between occurrences of a given discharge, based upon statistical analysis. The recurrence interval is simply the inverse of the Annual Exceedence Probability value. For example, a flood having a 1 percent annual exceedence probability (AEP) would happen, on average, every 100 years (1/0.01 AEP).

Regulatory Floodway: The floodplain area that is reserved in an open manner by federal, state, or local agency requirements (i.e., unconfined or unobstructed either horizontally or vertically) to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP).

Scour: This term refers to the displacement of soils (including rock) due to flowing water. It is usually used to describe the removal of channel substrate in the vertical direction, whereas "erosion" is the preferred term to describe lateral channel adjustments to flowing water. There are three forms of scour:

1. **Contraction Scour:** Contraction scour, in a natural channel or at a bridge crossing, involves the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow area at the bridge, which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel.
2. **Local Scour:** Removal of material from around piers, abutments, spurs, and embankments caused by an acceleration of flow and resulting vortices induced by obstructions to the flow.
3. **Long-Term Scour:** Aggradation and degradation of the stream bed. These terms refer to the general and progressive buildup (aggradation) or lowering (degradation) of the longitudinal profile of a channel bed due to sediment deposition or removal, respectively. The terms "deposition" and "sedimentation" are often used to describe "aggradation"; "incision" and "head-cutting" are often used to describe "degradation."

Thalweg: The line extending down a channel that follows the lowest elevation of the bed.

Hydraulic Design Criteria

The following summarizes DOT&PF's basic hydraulic criteria used for the design of bridge waterway openings:

1. **Design Flood Frequency**. The hydraulics engineer determines the minimum design flood frequency, which is based on the roadway classification, scour design criteria, flood plain regulations, and ranges from the 50-year event to the 500-year event.
2. **Maximum Allowable Backwater.** On FEMA-delineated floodways, no backwater may be introduced by the structure. On FEMA-delineated floodplains, 1 foot of maximum backwater may be introduced. For all sites, the maximum allowable backwater shall be limited to an amount that will not result in unreasonable damage to upstream property or to the highway. The hydraulics engineer will determine the allowable backwater for each site.
3. **Freeboard**. Establish a specified clearance, generally 3 feet, to allow for passage of debris. Evaluate clearance for the passage of ice or for icing conditions on a case-by-case basis, but is generally 3 feet or greater. Where this is not practical, establish the clearance based on the type of stream and level of protection desired. For navigation channels, establish a vertical clearance conforming to federal requirements based on normally expected flows during the navigation season.
4. **Scour**. The bridge foundation must not fail or be damaged for the scour design event of the 100-year and 500-year flood. Check lesser flood events if there are indications that less frequent events may produce significantly deeper scour than the 100-year flood.
5. **Riprap**. For riverine applications, DOT&PF typical practice is to place riprap to an elevation of not less than 1 foot above the design surface water elevation. If ice is a concern, it may be warranted to place riprap to a higher elevation. For coastal applications, such as shore protection provisions, consult DOT&PF coastal engineers for design support.
6. **Ice**. Consider both static and flowing ice in the hydraulic design.

Costs

Economy of construction is usually a significant consideration in determining spans, pier locations, and orientation. Initial construction costs are always a factor in the structural design of a bridge, but this is only one element of the total economic cost of a stream crossing system. There are hydraulic considerations, maintenance costs, and risks of future costs to repair flood damages that should also be factors in the decision on the number of piers, their location, orientation, and type.

Bridge vs Culvert

In some cases, the waterway opening for a highway-stream crossing can be accommodated by either a culvert or a bridge. Estimates of costs and risks associated with each will indicate which structural alternative should be selected. Figure 11-1 lists some of the advantages and disadvantages of bridges and culverts.

As a general rule of thumb, a design peak discharge of 1000 cfs or less favors a culvert; greater than 1500 cfs favors a bridge. Between 1000 cfs and 1500 cfs, either structure type may be appropriate.

The Statewide Hydraulics Engineer and regional hydraulics engineer will collaborate on the selection.

Abutments

The principal hydraulic concerns for abutments are orientation, extent of channel encroachment, and the need for scour protection provisions. Scour risks can usually be addressed by providing an adequate waterway opening and scour countermeasures. Orientation is usually the same as for adjacent piers.

Piers - Coordination

Locating piers in waterways is an interactive process among the bridge engineer, foundation engineer, and hydraulics engineer.

Initially, the hydraulics engineer determines the required channel geometry to meet the hydraulic criteria (e.g., maximum backwater for 100-year flood).

The bridge engineer determines the number and length of spans, types of piers, and low-chord elevation.

Bridges	
Advantages	**Disadvantages**
• Less susceptible to clogging with drift, ice, and debris. • Waterway increases with rising water surface until water begins to submerge superstructure. • Scour increases waterway opening. • Flowline is flexible. • Minimal impact on aquatic environment and wetlands. • Widening does not usually affect hydraulic capacity.	• Requires more structural maintenance than culverts. • Abutment fill slopes susceptible to erosion and scour damage. • Piers and abutments susceptible to failure from scour. • Susceptible to ice and frost formation on deck. • Bridge railing and parapets hazardous as compared to recovery areas. • Deck drainage may require frequent maintenance cleanout. • Buoyancy, drag, and impact forces are hazards to bridges. • Susceptible to damage from stream meander migration.

Culverts	
Advantages	**Disadvantages**
• More roadside recovery area can be provided. • Grade raises and widening projects sometimes can be accommodated by extending culvert ends. • Requires less structural maintenance than bridges. • Frost and ice usually do not form before other areas experience the same problems. • Capacity increases with stage. • Capacity can sometimes be increased by installing improved inlets. • Usually easier and quicker to build than bridges. • Scour is localized, more predictable, and easier to control. • Storage can be utilized to reduce peak discharge. • Avoids deep bridge foundations.	• Multiple barrel culverts, whose width is considerably wider than the natural approach channel, may silt in and may require periodic cleanout. • No increase in waterway as stage rises above soffit. • Can clog with drift, debris, or ice. • Possible barrier to fish passage. • Susceptible to erosion of fill slopes and scour at outlets. • Susceptible to abrasion and corrosion damage. • Extension may reduce hydraulic capacity. • Inlets of flexible culverts susceptible to failure by buoyancy. • Rigid culverts susceptible to separation at joints. • Susceptible to failure by piping and/or infiltration.

Figure 11-1
Bridge vs. Culvert (Hydraulic Considerations)

The hydraulics engineer evaluates the bridge design proposal to determine if it meets the hydraulic requirements of the waterway opening. For example, to meet the hydraulic criteria, it may be necessary to increase span lengths.

Next, the bridge engineer and foundation engineer evaluate potential foundation designs for the pier and provide preliminary design information to the hydraulics engineer for scour analysis. If the resulting foundation design is judged to be too costly, the bridge engineer will evaluate reducing the number of piers or eliminating piers altogether based on overall structure costs, environmental impacts, constructability, etc.

The highway profile (i.e., vertical alignment and bridge end elevations) is an additional highway design element in the iterative process to identify the number and location of piers in waterways. The profile can have a significant impact on the overall bridge opening and floodplain flow conditions. The roadway designer may prefer, for example, to lower the highway profile due to significant right-of-way impacts which, all other factors being equal, reduces the hydraulic capacity of the waterway opening and increases the frequency of overtopping.

Ultimately, all of these factors (i.e., structural, hydraulic, geotechnical, roadway, environmental, costs) must be evaluated to identify the optimum number and location of piers.

Piers - Hydraulic Design Considerations

Limit the number of piers in any channel to a practical minimum, and avoid piers in the channel of small streams, if practical. The cost of constructing piers increases with the water depth. Piers properly oriented with the flow do not usually contribute significantly to bridge backwater, but they can contribute to scour. Align piers with dominant flow direction at flood stage to reduce floating debris accumulation potential, to reduce the contraction effect of piers in the waterway, to minimize debris forces, and to minimize backwater and scour. Pier orientation is difficult where flow direction changes with stage or time. In this case, consider the use of a single column pier.

Pier shape is also a factor in local scour. Rounding or tapering the leading edges of piers helps to decrease the accumulation of debris and reduces local scour at the pier.

Foundations

The foundation is usually the bridge element that is most vulnerable to flood damage. Examination of boring logs and subsurface material information is important to the prediction of potential scour depths.

Driven piles or drilled shafts usually depend upon the surrounding material for skin friction and lateral stability. In some cases, they can be extended to rock or other dense material for load-carrying capacity through tip resistance. Determine tip elevations for piling or drilled shafts considering estimates of potential scour depths and bearing to avoid losing lateral support and load-carrying capacity during floods.

Consider the potential scour and the possibility of channel shifts in designing foundations for bridges on floodplains and spans approaching the stream channel. Do not consider the thalweg (i.e., the line or path connecting the lowest flow points along the channel bed) to be in a fixed location when establishing founding elevations. The history of a stream and a study of how active it has been can be useful in making decisions on pile and drilled shaft tip elevations.

11.2.3. Geotechnical Engineering

The foundation engineer in the Statewide Material Section provides preliminary foundation information for a proposed site. Bridges located at different sites have different foundation requirements, which must be considered in determining the bridge location.

11.2.4. Right-of-Way

Right-of-way and utilities can have a significant influence on the bridge location and the right-of-way acquisition and utility relocations can require a significant timeframe to complete. The Right-of-Way Section can provide estimates on cost, number of properties and utilities encountered, possible difficult acquisitions, and approximate time frames. In addition to property acquisition, most projects require temporary and permanent easements for construction staging areas, access, future maintenance, and construction.

Right-of-way issues are usually handled through, and coordinated by, the regional project manager, who will typically address right-of-way estimates and impacts. This information, in turn, will assist the bridge engineer in preparing the bridge layout within the right-of-way constraints. Plot the right-of-way lines on the General Layout and Site Plan sheets.

Consider the following right-of-way factors when selecting the structure type:

1. **Expensive Right-of-Way**. If right-of-way will be expensive, this may lead to the use of retaining walls and other measures to reduce right-of-way impacts.
2. **Structure Depth**. The available right-of-way at the bridge site may affect the vertical alignment of the structure which may, in turn, affect the acceptable structure depth to meet the vertical clearance requirements. The depth of the superstructure can be a significant issue in urban areas. Right-of-way acquisition costs are high, and roadway profiles cannot usually be raised due to access rights on approaches.

Bridge designs must be consistent with DOT&PF utility accommodation policies. Chapter 16 discusses utility attachments to bridges.

11.2.5. Environmental

Potential environmental impacts can significantly affect bridge location and structure-type selection and configuration, especially for highway bridges over streams. In general, all bridge projects must attempt to avoid and minimize environmental impacts as practical, especially in sensitive areas (e.g., wetlands, endangered species habitat, essential fish habitat, anadromous or resident fish streams). The DOT&PF regional environmental offices, in coordination with the DOT&PF statewide environmental office, are responsible for identifying all environmental resources and other related issues of concern in the project area and for coordinating with the Bridge Section (through the regional project manager) to evaluate the potential project impacts on these resources and issues. In addition, the regional environmental offices are responsible for ensuring that proposed projects comply with applicable requirements for public involvement. See the *Alaska Environmental Procedures Manual* for a detailed discussion on the environmental process, environmental impact issues and considerations, and environmental permit requirements.

Environmental Class of Action

For every DOT&PF project involving federal funding, authorizations, or approvals, the regional environmental office, in coordination with the statewide environmental office and the FHWA Alaska Division, will determine the Environmental Class of Action (i.e., the level of processing required for compliance with the National Environmental Policy Act [NEPA]). This determination will be based on the results of the evaluation of project impacts and the nature and scope of the proposed project, which will be one of the following:

1. **Categorical Exclusion**. A Categorical Exclusion (CE) is issued for categories of projects that do not individually or cumulatively have a significant effect on the environment and, therefore, do not require the preparation of an EA or EIS.
2. **Environmental Assessment**. An Environmental Assessment (EA) is prepared for projects for which the significance of the environmental impact is not clearly established.
3. **Environmental Impact Statement**. An Environmental Impact Statement (EIS) is prepared for projects where it is known that the action will have a significant effect on the environment.

Permits/Approvals

A proposed bridge project may precipitate the need for one or more environmental permits or approvals. The regional environmental office is responsible for coordinating with the applicable federal or state agency to acquire the applicable permit(s) or approval(s). This may require considerable coordination with the Bridge Section through the regional project manager. The following briefly discuss these permits/approvals.

US Army Corps of Engineers Section 404 Permit. The Section 404 Permit is required for the discharge of dredge or fill material into any waters of the United States, including wetlands. The purpose of Section 404 is to restore and maintain the chemical, physical, and biological integrity of the Nation's waters through the prevention, reduction, and elimination of pollution.

Section 401 Water Quality Certification. Pursuant to Section 401 of the *Clean Water Act*, the Section 401 Water Quality Certification is issued by the Alaska Department of Environmental Conservation (DEC) based on regulations issued by the US Environmental Protection Agency. The purpose of the Section 401 Certification is to ensure that a permitted discharge will not violate applicable water quality standards. A Section 401 Certification (or waiver of Certification) is required in conjunction with any federal permit (e.g., a Section 404 Permit) to conduct activities that may result in any discharge into waters of the United States.

Section 402 NPDES Permit. Pursuant to Section 402 of the *Clean Water Act*, the Section 402 National Pollutant Discharge Elimination System (NPDES) Permit is issued by DEC based on regulations issued by the US Environmental Protection Agency. The purpose of the Section 402 Permit is to restore and maintain the chemical, physical, and biological integrity of the Nation's waters through prevention, reduction, and elimination of pollution.

US Coast Guard Section 9 Permit. Pursuant to Section 9 of the *Rivers and Harbors Act* of 1899, the Section 9 Permit is issued by the US Coast Guard. The Section 9 Permit ensures that there will be no interference to navigation on the navigable waterways of the United States. The Permit is required for the construction, modification, replacement, or removal of any bridge or causeway over a navigable waterway. FHWA implementing regulations can be found at 23 CFR 650, Subpart H, Navigational Clearances for Bridges (23 CFR 650.801 and following).

Floodplain Encroachment Approval and Finding. Pursuant to Executive Order 11988 "Floodplain Management," DOT&PF must seek approval from the Federal Emergency Management Agency (FEMA) for any federally funded/regulated project that produces a significant floodplain encroachment. If a project will have a significant floodplain encroachment, the project will require either an Environmental Assessment (EA) or Environmental Impact Statement (EIS). A proposed action that includes a significant floodplain encroachment will not be approved unless FHWA finds (pursuant to 23 CFR650A) that the proposed action is the only practical alternative.

Fish Habitat Permits. Fish habitat permits may be required for DOT&PF construction projects. These permits are issued by the Alaska Department of Fish and Game (ADF&G).

Under the *Fishway Act*, an "840" permit must be obtained for work that may impede the efficient passage of fish. Generally, DOT&PF obtains this permit before placing hydraulic structures in streams with resident fish, but "840" authorization is also required for the following activities:

- stream diversions or realignments,
- low-water crossings, and
- removal of material below ordinary high water (OHW).

Under the *Anadromous Fish Act*, an "870" permit must be obtained for any construction activity within or across a specified anadromous waterbody that would use, divert, pollute, or change the natural flow or bed. Essentially, any construction activity that is below the OHW of a cataloged anadromous fish stream requires an "870" permit.

Special Area Permit. ADF&G administers Alaska Statutes for the protection of State of Alaska Refuges, State Sanctuaries, and State Critical Habitat Areas. These statutes require a "special area permit" from ADF&G for any habitat-altering work, including construction activity, in a designated state refuge, sanctuary, or critical habitat area.

Fish and Wildlife Coordination Act Consultation. This Act requires consultation with the US Fish and Wildlife Service (USFWS) and state fish and wildlife agencies where the "waters of any stream or other body of water are proposed or authorized, permitted, or licensed to be impounded, diverted, or otherwise controlled or modified" by any agency under a federal permit or license. The purpose of consultation is for "preventing loss of and damage to wildlife resources."

Hazardous Waste

The regional environmental offices are responsible for identifying and evaluating hazardous waste sites and for determining the needed mitigation measures. Four specific types of hazardous waste that may require treatment for a bridge project include:

1. **Paint Removal**. Removal of paint from steel bridges that may contain heavy metals or from concrete bridges that may contain asbestos.
2. **Fine Surface Finish**. This type of concrete finish may contain asbestos.
3. **Timber Removal**. Salvaging or disposing of treated timber from an existing bridge.
4. **Plates**. Asbestos blast plates on railroad overpasses.

11.3. Span Length and Configuration

The total required length of a bridge is usually fairly easy to determine. Determining the optimum number of spans can be more difficult. This depends upon the:

- roadway profiles;
- vertical clearances;
- construction requirements (e.g., river diversions, falsework openings);
- environmental factors;
- depth of structure;
- allowable locations of piers;
- foundation conditions;
- waterway opening requirements;
- safety of underpassing traffic;
- navigational requirements; and
- flood debris considerations.

See Section 11.5 for DOT&PF practices for span ranges of various superstructure types.

11.3.1. Waterway Crossings

Typically, place abutments for bridges crossing streams and rivers at the banks of the river so that the bridge does not affect flow. In addition, abutments can also be placed a sufficient distance back from the edge of the banks (outside the "ordinary high-water line") to keep excavations, backfill, and riprap out of the river, often easing environmental constraints. Many Alaska river systems have construction windows, endangered species, and water quality requirements that greatly restrict construction activities within the ordinary high-water boundaries.

Minimize the use of piers, because they add cost and construction time, and reduce the hydraulic opening. Each pier is assumed to collect debris during a flood, which further reduces the hydraulic opening and increases scour. However, more supports allow for a shallower superstructure depth. Streams and rivers almost always require deep foundations. A bridge with foundations that remain out of the water greatly reduces foundation costs and can in many cases be the least cost alternative. Access into a stream or river (usually through adjacent property), pile drilling and driving equipment logistics, river diversions, settling basin requirements, environmental restrictions, and risk of flooding greatly increase the cost of placing a support or multiple supports in a stream or riverbed.

See Section 11.2.2 for a further discussion on hydraulic considerations for bridge design.

11.3.2. Highway Crossings

Highway bridges over other highways ("overcrossings") shall have their abutments set based on the anticipated future width requirements of the highway beneath the bridge. The number of lanes and shoulder widths are based on 20-year traffic projections; however, the *LRFD Specifications* require a 75-year design life for bridges. Traffic projections to 75 years are highly speculative. Consider some provision for future widening beyond 20 years. For example, spill-through abutments can accommodate the future construction of a retaining wall to increase the width of roadway under the bridge.

11.3.3. Railroad Crossings

Figure 1130-1 of the *Alaska Highway Preconstruction Manual* presents vertical clearance requirements for railroad overheads. Horizontal clearances will vary depending on the obstruction type (i.e., crashworthy pier, non-crashworthy pier, embankment) and the possibility of future track expansion; therefore, written direction from the Alaska Railroad is required. Typically, three-span and single-span bridges are built over railroad facilities. In many cases, highway bridges over railroads must be designed at skews over 30 degrees. Coordinate these bridge designs with the Chief Bridge Engineer. See Chapter 24 for more information.

11.4. General Design Considerations

As discussed in this section, the bridge engineer must evaluate certain general design factors in the selection of the structure type and layout.

11.4.1. Definition of Terms

Substructure vs Foundation

The dividing line between substructure and foundation is not always clear. Traditionally, foundations include those elements of the substructure that are in direct contact with, and transmit loads to, the supporting rock or soil. The *Alaska Bridges and Structures Manual* uses this definition.

Substructure vs Superstructure

A similar difficulty exists in separating substructure and superstructure. This *Manual* will refer to the substructure as any component or element (not including the foundation) supporting the bearings. The superstructure then consists of the bearings and all of the components and elements resting upon them.

11.4.2. Live-Load Deflection Criteria

Reference: LRFD Articles 2.5.2.6.2 and 2.5.2.6.3

The *LRFD Specifications* state that the traditional live-load deflection criteria are optional for bridges both with and without sidewalks because static live-load deflection is not a good measure of dynamic excitation. DOT&PF does not require that the live-load deflection criteria be satisfied for final design. Nonetheless, these criteria provide a good starting point for preliminary design. The live-load deflection criteria of the *LRFD Specifications* are calibrated to yield comparable results for the HL-93 notional live-load model as the provisions of the *Standard Specifications for Highway Bridges* with the HS20-44 live-load model.

11.4.3. Jointless Bridges

Where practical, the preferred choice is multiple simple span decked bulb-tee girders tied together with cast-in-place pier diaphragms and semi-integral abutments forming a jointless bridge. Historically, problems with expansion joints include corrosion caused by deicing chemicals leaking through the joints and accumulation of debris and other foreign material restricting the free joint movement. This often results in joint damage, differential elevation at the joints causing additional impact forces, unexpected bridge movements and settlements that affect the joint, and high initial and maintenance costs.

The bridge engineer can eliminate joints with special consideration to:

- load path,
- gravity and longitudinal loads,
- effects of concrete creep and shrinkage and temperature variations,
- stability of superstructure and substructure during construction and service,
- skew and curvature effects,
- the superstructure-abutment-foundation connection design and details,
- effects of superstructure and substructure stiffness,
- settlement and earth pressure,
- effects of varying soil properties and type of foundation, and
- effects of the approach slab and its connection to the bridge.

Jointless bridges in service have demonstrated the ability to perform under the previous considerations. Therefore, in the absence of an in-depth analysis, it is reasonable to design a jointless bridge under the following parameters:

- 3 inches of total movement or less at the abutment, and
- 30 degree skew or less.

11.4.4. Number of I-Girders

Due to the advantages of redundancy, new girder bridges must have a minimum of four girders per span. In general, the cost of a girder bridge increases with the number of girders in the cross section. Conversely, structure redundancy increases with the number of girders. The basic objective is to identify a girder spacing and corresponding number of girders that optimizes the design of the superstructure by providing sufficient redundancy with minimal cost. In addition, consider the structural implications of maintaining traffic across the bridge during future operations to redeck or widen the bridge.

11.4.5. Interior vs Exterior Girders

Reference: LRFD Article 4.6.2.2.1

For economy of fabrication, design all girders within a span identically to the governing condition, either interior or exterior girder. However, detail the girder deck and shear keys of decked bulb-tee girders according to the girder placement in the cross section, either the interior or exterior girder.

11.4.6. Seismic Requirements

Reference: LRFD Articles 3.10, 4.7.4, 5.11, A10.1-3, and 11.6.5

The ability to predict the bridge displacements developed by earthquake motion is limited by the complexity of predicting the movement of the underlying earth material and the response of the structure. Incorporate the seismic requirements of the *Guide Specifications for LRFD Seismic Bridge Design* with the selection of a superstructure, substructure, or foundation type. The seismic demand and the period of the bridge are directly related. Therefore, the structure-type selection will dictate the earthquake resisting system (ERS), ductility capacity, and detailing requirements as specified in the *Guide Specifications*.

Ideally, bridges should have a regular configuration where plastic hinging is promoted in multiple, readily identifiable, and repairable yielding components. "Regular" bridges can usually be proportioned for gravity loads and then checked and detailed to resist seismic demands.

Although the *Guide Specifications for LRFD Seismic Bridge Design* provisions do not discuss preliminary structure-type selection, certain guidelines should be followed. In general, select the structure type with the following considerations:

1. **Alignment**. Minimize horizontal curvature.
2. **Substructure Skew**. Where practical, use substructures with little or no skew. Skewed supports cause rotational response with increased displacements.
3. **Joints**. Use continuous girders when practical.
4. **Foundations**. Do not use shallow foundations if the foundation material is susceptible to liquefaction.
5. **Substructure Stiffness**. Conform to the requirements of Sections 4.1.2 and 4.1.3 of the *Guide Specifications*.

11.4.7. Foundation Considerations

The following applies, in general, to shallow foundations:

Grade Adjustment

When considering structure-type selection, the ability to adjust the structure through jacking is an important issue. Jacking stiffeners or diaphragms may be required for steel superstructures. For bridges supported on shallow foundations, the subgrade may settle differently from the calculated estimates. It is understood that, where superstructures and substructures are integral with each other, this facilitation for adjustment cannot exist.

Consider the nature of the subgrade prior to the final selection and design of the superstructure, substructure, and foundation to ensure adjustability if needed.

Settlement Limits

Experience demonstrates that bridges can accommodate more settlement than traditionally allowed in design due to creep, relaxation, and redistribution of force effects. LRFD Article 10.6.2.2.1 mandates that settlement criteria be developed consistent with the function and type of structure, anticipated service life, and consequences of unanticipated movements on service performance.

11.4.8. Aesthetics

Reference: LRFD Article 2.5.5

Structures should be aesthetically pleasing. The *LRFD Specifications* emphasize and DOT&PF encourages the objective of improving the appearance of highway bridges. In bridge design, the Department promotes uninterrupted lines, contours that follow the flow of forces, and the avoidance of cluttered appearances.

Any bridge design must integrate three basic elements: efficiency, economy, and appearance. Regardless of size and location, consider the quality of the structure, its aesthetic attributes, and the resulting impact on its surroundings.

The bridge engineer should adhere to the following design guidelines for aesthetic treatments of bridges:

- Use a consistent bridge design
- Use simple substructure and support features
- Consider fill embankments, bridge rails, and approach rails as part of the bridge design
- Select vandalism-resistant finishes

- Create a visual design unity among all existing and new structures along a highway segment
- Integrate landscape and aesthetics at the onset of project planning

11.4.9. Construction

General

The bridge engineer must review and recommend (to the construction Project Engineer) the approval or rejection of all erection or construction plans required by the contract documents.

Access and Time Restrictions

Water-crossing bridges will typically have restrictions associated with their construction. Consider these during structure-type evaluation.

Regulations administered by various state and federal agencies may restrict the time period that the Contractor will be allowed to work within the waterway. Depending on the time limitations, a bridge with fewer piers or faster pier construction may be more advantageous even if more expensive.

Staged Construction

Occasionally, due to the proximity of existing structures or a congested work area, it may be necessary to build a structure in multiple stages. The arrangement and sequencing of each stage of construction is unique to each project, and the bridge engineer must consider the requirements for adequate construction clearances and the requirements of the traveling public. If staged construction is required, include a staging sequence and controlling lane/construction dimensions in the contract documents.

Construction Costs

Initial construction cost is one factor in the selection of the structure type, but not the only factor. Also consider future expenditures during the service life of the bridge. The initial costs depend on a variety of factors including:

- type of structure
- economy of design
- market conditions
- experience of local contractors
- fabrication shop location
- local availability of structural materials and labor

Falsework

Falsework is an expensive construction item. If the bridge is over a waterway, will have a high finished elevation, or both, the cost of the falsework may become prohibitive, and the bridge engineer must consider other structural systems. The following will apply to the use of falsework:

1. **Railroads**. The Alaska Railroad Corporation (ARRC) has its own requirements for falsework over its facilities. Contact ARRC early in project development to determine if falsework may be used and ARRC's minimum clearance requirements. See Chapter 24 for more information.
2. **Environmental**. Some sites may be very sensitive environmentally, and the use of falsework may be prohibited.
3. **Hydraulics**. For falsework over a waterway, the hydraulics engineer will provide the minimum falsework opening dimensions.
4. **Traffic Impacts**. Constructing falsework over traffic poses a number of risks. Installing and removing falsework requires extended lane closures or expensive traffic crossovers. Vehicular impacts to falsework can pose a hazard to the traveling public and construction workers.

11.4.10. Maintenance and Durability

The structure-type selection will, over the life of the structure, have a major impact on maintenance costs. Based on type of material, the following is the approximate order of desirability from a maintenance perspective:

1. prestressed concrete
2. metalized steel

The following maintenance considerations apply:

1. **Deck Joints**. Open, or inadequately sealed, deck joints are the foremost reason for corrosion of structural elements by permitting the leakage of salt-laden water through the deck. To address this, the *LRFD Specifications* promote jointless bridges with integral or semi-integral abutments, continuous decks, and improvements in drainage.

2. **Bridge Inspection**. In addition to the maintenance needs of the structure, consider the bridge inspection logistics including access. Provide 3 feet minimum head room and a 3-foot wide berm in front of the abutments.

3. **Structural Details**. As another maintenance/inspection consideration, as practical, limit the number of articulated structural details (e.g., bearings, expansion joints).

11.4.11. Future Widening

Consider the possibility of future structure widening. For example, structures supported by single columns or cantilevered piers cannot practically be widened; a separate adjacent structure will be required.

Almost every superstructure type can be widened, but not with the same level of ease. Deck-on-girder bridges and systems consisting of prefabricated elements lend themselves to widening.

11.5. Superstructures

This section discusses those factors that must be considered in the selection of the superstructure type in preliminary design.

11.5.1. General Considerations

Throughout the nation, many types of superstructures have been developed for the myriad applications and constraints that prevail at bridge sites. However, DOT&PF, like other state DOTs, has narrowed its selection of superstructure types to a relatively small number based on DOT&PF's experience, geography, terrain, environmental factors, construction logistics, local costs, local fabricators, the experience and skills of the contracting industry, availability of materials, and DOT&PF preference. Preferred superstructure types promote uniformity throughout the state and simplify the bridge design process.

Substructure/Foundation Type Considerations

The selection of the foundation type typically occurs after the superstructure type selection. See Section 11.7. Therefore, the bridge engineer must anticipate the nature of the foundation characteristics in selecting the type of superstructure. Consider the following:

- **Number of Supports**. The expected foundation conditions will partially determine the number of and spacing of the necessary substructure supports. This will have a significant impact on the acceptable span lengths.
- **Scour**. The geologic or historic scour may have a significant impact on the foundation design which may, in turn, have a significant impact on the superstructure-type selection.

11.5.2. Precast, Prestressed Concrete, Decked Bulb-Tee Girder

Description

Precast, prestressed concrete, decked bulb-tee girders are standardized precast, prestressed T-girders with depths in 12-inch increments starting at 3′-6″ deep. The girders have top flanges that act as the deck through the transverse placement of the girder flange-tip to flange-tip. A membrane and asphaltic concrete overlay typically completes the deck surface.

See Chapter 14 for a detailed discussion on DOT&PF design practices for decked bulb-tee girders.

Typical Usage

Due to its excellent adaptability to the constraints in Alaska, the decked bulb-tee girder is the most common type of superstructure used by DOT&PF. During the last ten years, approximately 80% of the new bridges constructed in Alaska have been this type. In the absence of mitigating factors, DOT&PF uses the decked bulb-tee girder for all spans up to 145 feet on the state highway system and for rural applications where adequate lifting capacity is available for girder erection.

The decked bulb-tee girder bridge can only be used on nearly tangent alignment. As a general guide, decked bulb-tee girders can accommodate a 1-foot chord offset using curved flanges. For larger offsets, a wider bulb-tee girder bridge or another superstructure type (e.g., steel or precast concrete girders with a cast-in-place concrete deck) is a better choice.

Advantages/Disadvantages

Advantages of this structure type include the elimination of the need for a cast-in-place, reinforced concrete deck, moderate to low construction cost, low maintenance cost, no falsework, and fast on-site construction. Its disadvantages include limited ability to adapt to complex geometrics, limited span lengths, and relatively heavy member weight. Precast, prestressed concrete girders require careful handling during transportation and erection. Also, decked girders are heavier than girders without a deck.

Typical Girder Spacing

Because decked bulb-tee girders are placed flange-tip to flange-tip to form the roadway surface area, the girder spacing is equal to the top flange width, which is typically 5 feet to 7 feet.

11.5.3. Special Application Superstructure Types

Composite Steel I-Girders

Description. Composite steel I-girders can be either rolled shapes with spans up to approximately 125 feet or plate girders with spans up to approximately 250 feet. Plate girders can have a constant or variable depth. Abrupt depth changes are not considered aesthetically pleasing. Continuous girders can also be deepened (haunched) at the supports and reduced at the center of the span where vertical clearance is tight. These haunched girders usually have a parabolic variation in depth.

Most structural steel is fabricated out of state, which increases the cost of this structure type. Girder field sections can be easily transported in lengths up to approximately 90 feet. Longer girder segment lengths may be feasible on a case-by-case basis up to 150 feet. Use bolted splices to construct single girder lines up to approximately 1000 feet in length. Consider how this structure type will be erected, where the erection crane(s) will be located, and how the girders will be delivered to the site.

This is a deck-on-girder type of structure; therefore, a cast-in-place or precast deck is required, but the deck can be removed if needed in the future without adversely affecting the steel I-girders. This structure type can also be used with large skews and on horizontal curves, but minimize these geometric features in high seismic regions.

See Chapter 15 for a detailed discussion on DOT&PF design practices for structural steel superstructures.

Typical Usage. DOT&PF typically limits the use of structural steel superstructures to sites where a decked bulb-tee girder superstructure will not work for geometric, environmental, cost, or constructability reasons. In addition, steel girders are used where, in rural applications, bulb-tee girders are too heavy to transport and erect.

If a rolled-beam design is proposed for a new bridge, the contract documents should allow the substitution of a welded plate girder with equivalent plate dimensions at the Contractor’s discretion.

Consider the transportation of girders when identifying field splice locations.

Advantages/Disadvantages. When compared to other superstructure types, advantages of composite steel I-girders include relatively simple details and formwork, adaptable to complex geometrics, replaceable decks, and long-span capability. The structural characteristics for composite steel I-girders provide relatively low dead load and, therefore, may be more suitable when foundation conditions are poor.

The disadvantages of composite steel I-girders include moderate to high construction costs, high maintenance costs, and attention to detailing practices. Poor detailing will greatly increase the cost of the bridge and can decrease durability through fatigue cracking. Composite steel I-girders have a higher maintenance cost than concrete bridges. Steel girders are relatively flexible without the bracing provided by the diaphragms and concrete deck, and they require careful handling during transportation and erection.

Typical Girder Spacing. Girder spacings for steel I-girders are normally from 6 feet to 9 feet. Deep plate girder sections benefit the most from wide girder spacings. Shallow plate girders and rolled beams do not accommodate wider girder spacings and may require spacings less than 8 feet when at the limit of the depth-to-span ratios. The bridge engineer can control the design of shallow girder sections by deflection requirements.

Composite Steel Tub Girders

Description. Composite steel tub girders are plate girders with two webs with a common bottom flange. The webs are usually inclined to improve aesthetics and reduce the width of the bottom flange. Spans are economical from about 150 feet up to approximately 300 feet. Steel tub girders can have a variable depth, but this significantly increases the cost of the bridge. They can also be used on very tight-radius curves due to their high torsional stiffness. They do not, however, adapt well to skews or variable widths. Consider fabrication, transportation, and erection of this structure type. Steel tub girders are difficult to handle in the shop due to their size and weight. They require significant bracing during fabrication and erection. Stay-in-place steel forms can provide an innovative top-lateral bracing solution and have been employed by the Department in the past. In addition, steel tub girders are susceptible to thermal movements once erected and require temporary external bracing between boxes.

Typical Usage. For sites where the decked bulb-tee girder is not appropriate, composite steel tub girders are favored over steel I-girders because of their better stability during erection. In addition, composite steel tub girders are also considered in urban areas where a steel-I girder could be used but enhanced aesthetics are desired. However, tub girders are typically not practical in remote areas because of the relatively high girder weight.

Advantages/Disadvantages. Advantages of composite steel tub girder bridges include fast on-site construction, adaptability to tight-radius curves, replaceable deck, and longer span capability. Higher torsional resistance makes it desirable on horizontally curved alignments. Closing the girder section using stay-in-place forms makes the torsional resistance available during erection, increasing girder stability during transportation and erection.

Disadvantages include the high construction costs and not being readily adaptable to skewed or variable-width bridges. Composite steel tub girders require complicated fabrication, welding, and erection. This structure type has higher maintenance cost than concrete bridges.

Typical Girder Spacing. Web spacings are normally 5 feet to 10 feet, resulting in girder spacings of 10 feet to 20 feet. Deep sections benefit the most from wider web spacings. Shallow sections do not accommodate wider girder spacings.

Precast, Prestressed Concrete Bulb-Tee Girders

Description. Precast, prestressed concrete bulb-tee girders are similar to the standard decked bulb-tee girders but with a standard top flange width. A separate deck, either cast-in-place or precast concrete, is required to form the roadway. The practical limitation on span length is 150 feet, although other state DOTs have used span lengths of up to 215 feet. This is a deck-on-girder type of structure; the deck can be removed if needed without adversely affecting the girders. It does not adapt well to large skews and cannot be used on tight horizontal curves (e.g., radii less than 500 feet) or bridges with a variable width.

Typical Usage. Precast concrete girders are generally only used for span lengths up to 145 feet where roadway geometry (i.e., horizontal curvature) dictates the use of a CIP deck and a structural steel system has significant disadvantages. Consider the transportation of girders in selecting girder lengths.

Advantages/Disadvantages. Advantages of this structure type include moderate construction cost, low maintenance cost, replaceable deck, and moderately fast on-site construction. Its disadvantages include the need for a cast-in-place deck, limited span lengths, and slightly higher depth-to-span ratios. Precast, prestressed concrete girders require careful handling during transportation and erection.

Typical Girder Spacing. Girder spacings are normally from 5.5 feet to 7 feet. Concrete strength and the number of prestressing strands usually control the girder spacing.

11.5.4. Superstructure Types Used With Approval

The bridge engineer can consider superstructure types other than the decked bulb-tee girder and special application types may be used, including:

- post-tensioned, cast-in-place box girders,
- steel truss,
- steel-tied arch, and
- cable-supported.

The bridge engineer must investigate the experience of other owners, and the acceptability of these superstructure types must be based upon their successful experiences. The Chief Bridge Engineer must approve the selection of these structure types.

11.6. Substructures

11.6.1. Objective

This section discusses the types of substructure systems used by DOT&PF and their general characteristics. Use this guidance to select the substructure type that is suitable at the site to economically satisfy the geometric and structural requirements of the bridge and to safely use the strength of the soil or rock to accommodate the anticipated loads. Chapter 18 discusses the detailed design of substructure elements.

11.6.2. Abutments

Reference: LRFD Article 11.6

General

Abutments can be classified as flexible or rigid. Flexible abutments transmit earth pressures on the abutments through the superstructure eliminating expansion joints at the end of the superstructure. Semi-integral abutments are flexible abutments. Rigid abutments incorporate expansion joints at the end of the bridge to accommodate thermal movements. Seat-type abutments are rigid abutments. Flexible abutments must be able to accommodate the movements through elastic behavior of the bridge and the surrounding soil because the deck and girders are integral with the abutment. Flexible abutments are considered as pin-ended, expansion bearings in the superstructure analysis. Rigid abutments can be fixed or expansion based upon the choice of bearings.

Abutments may be further classified as either open or closed. Spill-through (or open) abutments are used for most bridges and are placed at the top of the slope. Bridge slopes are typically 2H:1V and based on stability requirements and erosion control.

Closed abutments are used when span lengths need to be reduced. Closed abutments are either a seat abutment or a MSE wall used as an abutment. For closed abutments, there are no fill slopes under the bridge but extensive retaining walls must be constructed. These retaining walls run either along the approaches to the bridge or parallel to the abutment. Retaining walls along the approaches are preferred from a visual perspective. Place closed abutment footings below the level of the highway running beneath the bridge resulting in tall exposed abutment faces.

Spill-through abutments result in longer spans compared to closed abutments, but the total cost is often less compared to closed abutments because spill-through abutments typically have less height. In general, spill-through abutments are considered more aesthetic, provide better sight distance, and more naturally accommodate trapezoidal channel geometry than closed abutments.

Basic Types

Use one of the following basic abutment types, in descending order of preference:

1. semi-integral abutments,
2. seat abutments,
3. MSE walls, and
4. integral abutments.

In general, semi-integral abutments are the preferred abutment type for single and multiple span decked bulb-tee girder bridges and short-span steel girder bridges. For decked bulb-tee bridges with abutment thermal movement greater than about 3 inches, seat-type abutments are typically required.

Semi-Integral Abutments

In semi-integral abutments, the superstructure is extended directly into the abutment backwall. There is no joint in the bridge deck, there may or may not be a pinned connection between the backwall and the pile cap, and there are bearings under the girders.

Floating abutments, where there is no connection between the diaphragm and the pile cap, are frequently used. Usually in a single span bridge, one end (typically the downhill one) will be pinned and the other end free. In a multispan bridge, both abutments will usually be free with fixity provided at the pier(s).

Pinned semi-integral abutments require flexible foundation elements to allow superstructure rotation and thermal motion. Typically, a single row of H-piles will provide the required flexibility for a pinned abutment, but drilled shafts, pipe piles, or spread footings will not. Typically, H-piles are oriented in the strong direction (flanges parallel to the long axis of the cap beam) to provide for greater deformation capacity prior to yielding.

The bridge engineer can use expansion bearings to reduce translation in the substructure.

Seat-Type Abutments

Seat-type abutments consist of a cap beam (supported by piles or columns), seat, and backwall. The superstructure is supported by bearings on the abutment seat. The backwall retains the backfill from the seat up so that the backfill is not in contact with the superstructure.

MSE Walls

Chapter 21 presents DOT&PF criteria on Mechanically Stabilized Earth (MSE) walls and discusses the respective responsibilities of the wall manufacturer and the DOT&PF in their design and construction. The use of an MSE wall as an abutment is a special application of this structural feature. DOT&PF has used two basic conceptual types:

1. **True Abutment**. An abutment supported by an MSE wall, in which the wall rests on a spread footing on top of reinforced earth.
2. **False Abutment**. A pile-supported abutment, in which the MSE wall wraps around an otherwise open abutment.

Do not use MSE wall abutments for bridges crossing waterways. Section 21.2 discusses the use of MSE walls in more detail. Section 18.1.5 discusses the use of MSE-wall abutments. Do not use true abutments in multi-span bridges with piers supported on deep foundations due to potential differential settlement concerns.

Integral Abutments

DOT&PF rarely uses integral abutments because of the inherent incompatibility with frozen ground and extreme thermal ranges. Only consider integral abutments where seasonal freeze of the soil is rarely anticipated. The Chief Bridge Engineer must approve their use.

11.6.3. Piers

Reference: LRFD Article 11.7

General

Piers consist of a pier cap supported on columns, pile extensions, or a pier wall. Extending a deep foundation above ground level to the superstructure, forming a pile extension bent, enhances the economy of substructures.

Pier Caps

Pier caps are usually reinforced concrete members that transfer girder loads into columns, pile extensions, or pier walls. In all cases, use a pier cap. These can be integral with the superstructure, drop (non-integral), or outrigger caps. Drop caps are the most common.

Integral caps may be used with either cast-in-place concrete, precast concrete, or steel girders. Integral caps used with steel girders can be either steel cross girders, cast-in-place concrete, or post-tensioned concrete. Only use an integral cap when necessary. Integral steel caps are often non-redundant, expensive and require precise fabrication. Integral concrete caps with steel girders are difficult to construct, usually require temporary falsework, and do not allow inspection of the top tension flanges after the bridge goes into service.

Use outrigger caps where a column support must extend beyond the edge of the superstructure. Do not use outrigger caps unless necessary. They should be simple spans with pin connections at the top of the columns. Pin connections reduce the torsional shear forces in the outrigger cap. Most outrigger caps are integral concrete and post-tensioned to reduce their depth and to control cracking.

General Usage

Columns, pile extensions, and pier walls are substructure components that support the cap. Use either single or multiple columns depending upon the width of cap and skew of the bridge. For pile extension piers, use either 2, 3, or 4-foot diameter pipe piles. Column sizes for other piers are typically not less than 3 feet in diameter.

The following summarizes DOT&PF typical practice for the selection of a pier type for bridges based on the type of crossing:

1. **Water Crossings**. If the bridge is less than 30 feet above the groundline, a pile extension bent is preferred. Otherwise, use a drilled-shaft, single-column pier, or a wall pier if ice forces require greater lateral resistance.
2. **Railroad Crossings**. If the pier is within 25 feet of the track centerline or future track centerline, use a solid pier wall that satisfies AREMA requirements.

3. **Highway Grade Separation**. Use pile extension bents where the bridge is less than 30 feet above the groundline, or use single columns supported on drilled shafts for a taller bridge.

The following briefly discusses design issues for the piers used by DOT&PF.

Pile Extension Bents

Pile extension bents result in minimal environmental impacts.

Hammerheads (Single-Column Piers)

Single-column or hammerhead piers are typically used where pile extension bents are not feasible and drilled shaft foundations are indicated.

Columns will have circular cross sections, unless a circular column cannot be designed for the required loading. In this case, use an oblong cross section.

The effects of skew can be eliminated by rotating the hammerhead relative to the crossing.

Multi-Column and Wall Piers

Multi-column and wall piers are typically used where spread footing foundations are feasible.

11.7. Foundations

11.7.1. Foundation Type Selection (Overview)

The selection of a foundation type involves an evaluation of the load/structural considerations for the bridge, the geotechnical factors pertaining to the native soils and rock at the site, and where the bridge traverses a waterway, the hydraulic characteristics related to the potential scour. As a starting point, the following presents general DOT&PF practices for selecting a foundation type:

- If the depth to bedrock is less than 10 feet, a spread footing may be the best choice.
- If the height of the bridge pier is less than 30 feet, a pile extension bent is preferred.
- If site conditions do not match either of the above, use a drilled shaft.

In addition, the following lists some of the site-specific factors that should be considered:

- driveability (high-impedance piles),
- constructibility,
- load-carrying requirements,
- scour-susceptibility,
- costs, and
- resource agency permits required.

11.7.2. Structural Foundation Engineering Report (SFER)

Section 17.2 discusses the content and format of the SFER, including the selection of a foundation type.

11.7.3. Types/Usage

The following summarizes DOT&PF's typical practices for the selection of the type of foundation. Chapter 17 discusses the detailed design of foundations.

Driven Piles

In general, DOT&PF uses driven piles where competent rock is greater than 10 feet below design grade and a short pier (height less than 30 feet) is required. In most cases, the best foundation/substructure selection at the site is a driven pile (with a pile extension bent as the supporting pier). Lateral foundation demands may require pile rock sockets. If rock sockets are required, consider drilled shaft foundations as a possible alternative.

If underlying soils cannot provide adequate bearing capacity or tolerable settlements for spread footings, use piles to transfer loads to deeper suitable strata through skin friction and/or point bearing. The selected type of pile is determined by the required bearing capacity, length, soil conditions, and economic considerations. DOT&PF primarily uses steel H-piles or steel pipe piles. See Chapter 17 for more information.

Spread Footings

Reference: LRFD Article 10.6

A spread footing is a shallow foundation consisting of a reinforced concrete slab bearing directly on the founding stratum. At sites with competent soil, spread footings provide a cost-effective foundation. The spread footing geometry is determined by structural requirements and the characteristics of supporting components, such as soil or rock. Their primary role is to distribute the loads transmitted by piers or abutments to suitable soil strata or rock at relatively shallow depths.

The use of spread footings requires firm bearing conditions; competent material (i.e., bedrock) must be exposed or near the ground surface (i.e., a maximum of 10 feet below the ground line). Spread footings are sometimes used where dense glacial till is near the surface and dry. They are not typically used at stream crossings where they may be susceptible to scour and at sites prone to liquefaction. Spread footings are often the best choice for retaining walls, sound barriers, etc.

Settlement criteria need to be consistent with the function and type of structure, anticipated service life, and consequences of unanticipated movements on service performance. Do not allow longitudinal angular distortions between adjacent spread footings greater than 0.008 radians in simple spans and 0.004 radians in continuous spans.

Drilled Shafts

Reference: LRFD Article 10.8

Drilled shafts are the most costly foundation alternative. A drilled shaft (also called a caisson) is a long, slender deep foundation element constructed by excavating a hole with auger equipment and placing concrete, with reinforcing steel, in the excavation. Casing and/or drilling slurry may be necessary to keep the excavation stable.

In general, DOT&PF uses a drilled shaft at bridges more than 30 feet in height (i.e., where a pile extension bent is not feasible) and where a spread footing/pile-supported footing is not feasible for constructability or structural reasons. Constructability concerns are where there are limits on in-stream work or tight construction zones, where driven piles are not economically viable due to obstructions to driving, or where limitations on vibration or construction noise exist. Structural concerns are where driven piles are not economically viable due to high loads and excessive slenderness (height-to-diameter ratio is too great).

11.8. Roadway Design Elements

11.8.1. Coordination

In general, the roadway design criteria will determine the proper geometric design of the roadway, and the bridge design will accommodate the roadway design across any structures within the project limits. This will provide full continuity of the roadway section for the entire project. This process requires proper coordination between the bridge engineer and roadway designer to identify and resolve any inconsistencies.

Initially, the roadway designer sets the geometrics, which are based on the *Alaska Highway Preconstruction Manual*. Check the proposed geometric design (e.g., clearances, horizontal curves, superelevation transitions, vertical curves, longitudinal slope, roadway approach, cross slopes, widths) to identify any modifications that may be warranted to better accommodate structural design considerations. Communicate any proposed modifications to the roadway designer, who will make the final decision on their incorporation.

11.8.2. Vertical Clearances

Figure 1130-1 in the *Alaska Highway Preconstruction Manual* provides the basic vertical clearance criteria adopted by DOT&PF. In addition, where structures with a prescribed vertical underclearance are constructed on spread footings, make allowance for any future settlement that would decrease the clearance. For structures founded in the approach fills, the allowance is 3 inches, unless a greater amount of settlement is expected.

DOT&PF policy is to provide an 18-foot vertical clearance for highways beneath railroad bridges for the Port of Anchorage to North Slope corridor to accommodate overheight loads.

11.8.3. Structure Length Calculations

The overall structure length is measured from the earth side face of the abutment to the earth side face of abutment. The following figures present criteria for determining structure length:

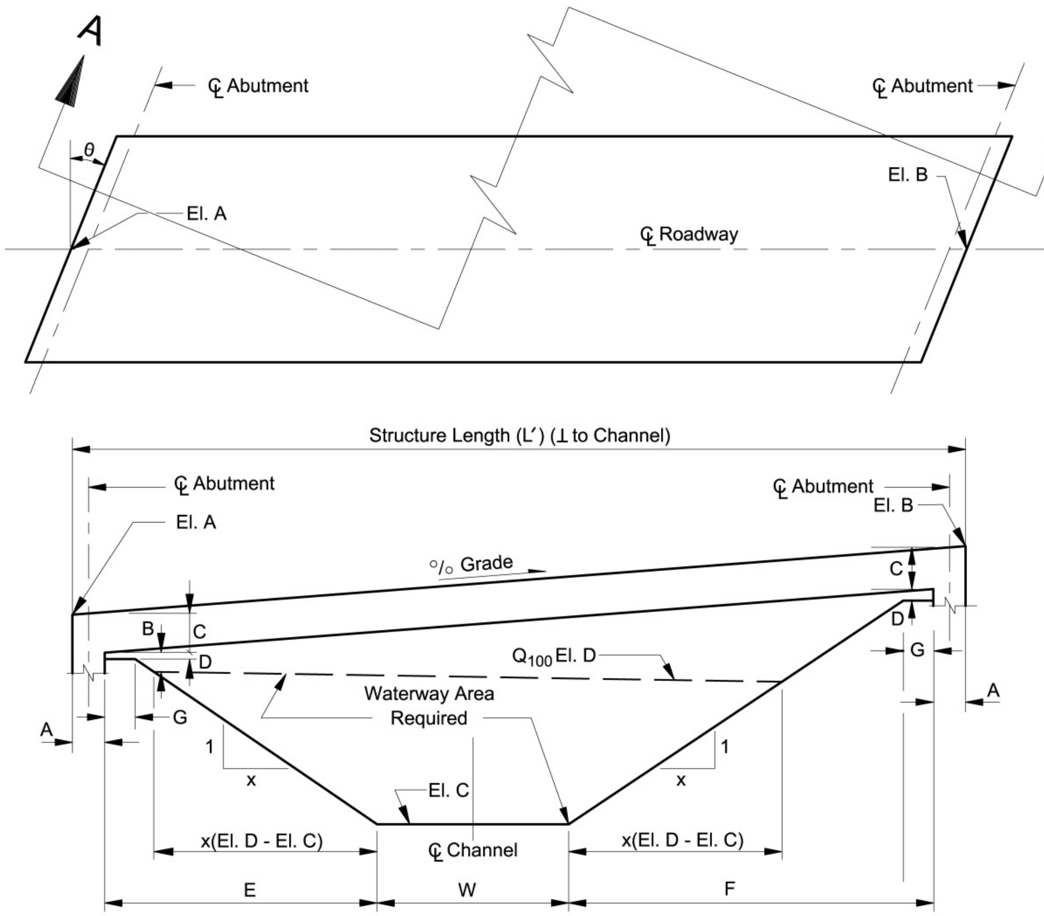
- Figure 11-2 "Structure Length for Stream Crossings"
- Figure 11-3 "Structure Length for Highway Crossings"

See Chapter 24 for highway bridges over railroads.

The major variables that determine the structure length are:

- the use of a spill-through abutment or seat abutment;
- seat width;
- for spill-through abutments, the slope;
- for waterway crossings, the waterway opening dimensions;
- for highway crossings, the width of the underpassing roadway cross section;
- roadway centerline grade; and
- the skew angle of the bridge.

The following figures assume that the bridge is on tangent and on a constant longitudinal gradient. The presence of a horizontal curve and/or a vertical curve will increase the length of the structure.



SECTION A-A
(Perpendicular to Channel)

θ = Angle of skew
A = Abutment width
B = Freeboard
C = Anticipated depth of superstructure
D = Distance from bottom of superstructure to top of abutment slope
E = (x) (El. A – C – D – El. C) + G
F = (x) (El. B – C – D – El. C) + G
G = Berm width
W = Width of channel (perpendicular to channel)
El. A = Elevation of top of deck
El. B = Elevation of top of deck
El. C = Bottom of channel elevation
El. D = Elevation of water surface at Q_{100}

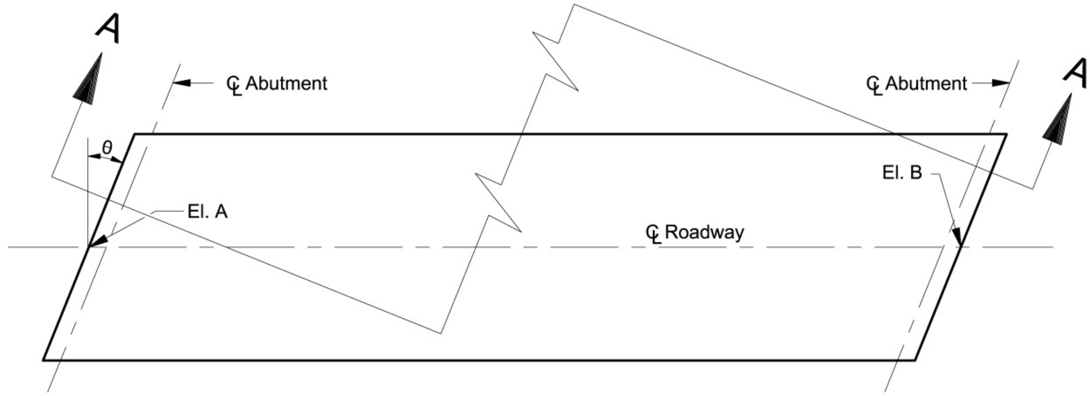
L′ = Structure length perpendicular to channel

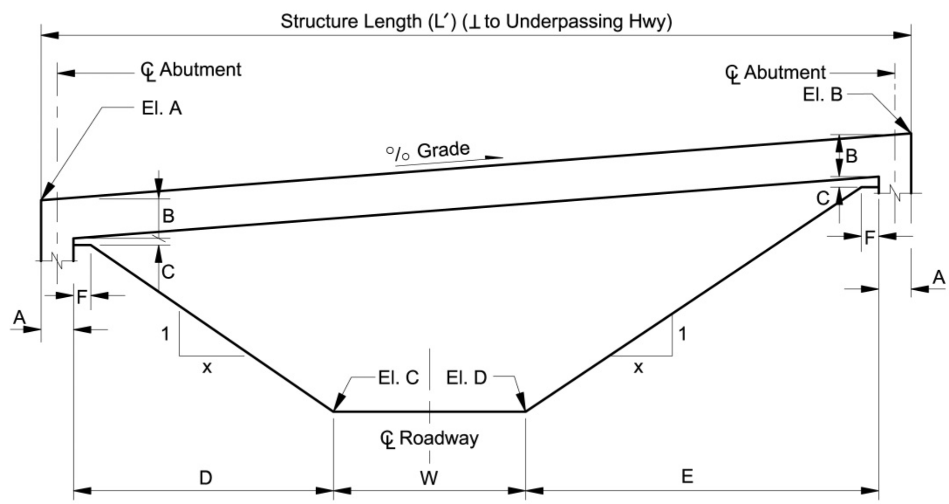
L = Structure length along ℄ roadway

L′ = A + E + W + F + A

L = L′/cos θ

Figure 11-2
Structure Length for Stream Crossings





SECTION A-A
(Perpendicular to Underpassing Hwy)

θ = Angle of skew
A = Abutment width
B = Anticipated depth of superstructure
C = Distance from bottom of superstructure to top of abutment slope
D = (x) (El. A – B – C – El. C) + F
E = (x) (El. B – B – C – El. C) + F
F = Berm width (3′ typical)
W = Width of underpassing roadway section
El. A = Elevation of top of deck
El. B = Elevation of top of deck
El. C = Elevation of toe of slope
El. D = Elevation of toe of slope

L′ = Structure length perpendicular to under-passing highway

L = Structure length along ℄ roadway

$L' = A + D + W + E + A$

$L = L'/\cos\theta$

Figure 11-3
Structure Length for Highway Crossings

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12. Loads and Load Factors

12.1. General

12.1.1. Load Definitions

Reference: LRFD Article 3.3.2

Permanent Loads

Reference: LRFD Article 3.5

Permanent loads are loads that are always present in or on the bridge and do not change in magnitude during the life of the bridge.

Transient Loads

Transient loads are loads that are not always present in or on the bridge or change in magnitude during the life of the bridge.

12.1.2. Limit States

Reference: LRFD Article 1.3.2

The *LRFD Specifications* group the traditional design criteria together within generalized groups of design criteria termed "limit states." The *LRFD Specifications* assign multiple load combinations to the various limit states.

Basic LRFD Equation

Design the components and connections of a bridge to satisfy the basic LRFD equation for all limit states:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \quad \textit{(LRFD Eq.1.3.2.1-1)}$$

Where:

γ_i = load factor
Q_i = load or force effect
ϕ = resistance factor
R_n = nominal resistance
η_i = load modifier as defined in LRFD Equations 1.3.2.1-2 and 1.3.2.1-3

The left-hand side of LRFD Equation 1.3.2.1-1 is the sum of the factored load (force) effects acting on a component; the right-hand side is the factored nominal resistance of the component for the effects. Consider all applicable limit-state load combinations for the Equation. Similarly, the Equation is applicable to both superstructures and substructures.

For the strength limit states, the *LRFD Specifications* are a hybrid design code in that, for the most part, the force effect on the left-hand side of the LRFD Equation is based upon elastic structural response, while resistance on the right-hand side of the Equation is determined predominantly by applying inelastic response principles. The *LRFD Specifications* have adopted the hybrid nature of strength design on the assumption that the inelastic component of structural performance will always remain relatively small because of non-critical redistribution of force effects. Ensure this non-criticality by providing adequate redundancy and ductility of the structures.

Load Modifier

The load modifier η_I relates the factors η_D, η_R, and η_I to ductility, redundancy, and operational importance. The location of η_I on the load side of the LRFD Equation may appear counterintuitive because it appears to relate more to resistance than to load. η_I is on the load side for a logistical reason. When η_I modifies a maximum load factor, it is the product of the factors as indicated in LRFD Equation 1.3.2.1-2; when η_I modifies a minimum load factor, it is the reciprocal of the product as indicated in LRFD Equation 1.3.2.1-3. These factors are somewhat arbitrary; their significance is in their presence in the *LRFD Specifications* and not necessarily in the accuracy of their magnitude. The LRFD factors reflect the desire to promote redundant and ductile bridges.

In general, use η_I values of 1.00 for all limit states, because bridges designed in accordance with this *Manual* will demonstrate traditional levels of redundancy and ductility. Rather than penalize less redundant or less ductile bridges, the DOT&PF does not encourage such bridges. DOT&PF may on a case-by-case basis designate a bridge to be of special operational importance and specify an appropriate value of η_I. For structural systems with only two longitudinal main members (e.g., two-girder/truss/arch bridges), η_I shall be taken as 1.20 for the girder/truss/arch.

Do not confuse the load modifier, η_I, accounting for importance of LRFD Article 1.3.5 with the categories of critical or essential bridges for seismic design of Article 3.1 of the *Guide Specifications for LRFD Seismic Bridge Design*. Use 1.0 for the importance load modifier used in the basic LRFD Equation, but use the critical or essential category to determine the minimum seismic requirements.

12.1.3. Load Factors and Combinations

Reference: LRFD Article 3.4.1

LRFD Table 3.4.1-1 provides the load factors for all of the load combinations of the *LRFD Specifications*.

Strength Load Combinations

The *LRFD Specifications* have calibrated the load factors for the Strength load combinations based upon structural reliability theory, which represents the uncertainty of their associated loads. The following simplifies the significance of the Strength load combinations, and it provides guidance on which Strength limit states are applicable to the bridge under design:

1. **Strength I Load Combination**. This load combination represents random traffic and the heaviest truck to cross the bridge in its 75-year design life. During this live-load event, a significant wind is not considered probable.

2. **Strength II Load Combination**. In the *LRFD Specifications*, this load combination represents an owner-specified permit load model. This live-load event has less uncertainty than random traffic and, thus, a lower live-load load factor. DOT&PF does not specify a design permit load. Therefore, this load combination is not applicable in Alaska.

3. **Strength III Load Combination**. This load combination represents the most severe wind during the bridge's 75-year design life. During this event, assume that no significant live load crosses the bridge.

4. **Strength IV Load Combination**. This load combination represents an extra safeguard for bridge superstructures where the unfactored dead load exceeds seven times the unfactored live load. Thus, the only significant load factor is the 1.25 dead-load maximum load factor. For additional safety, and based on engineering judgment, the *LRFD Specifications* has arbitrarily increased the load factor for DC to 1.5. This load combination typically governs only for longer spans, greater than approximately 200 feet in length. Thus, this load combination will only be necessary in relatively rare cases.

5. **Strength V Load Combination**. This load combination represents the simultaneous occurrence of a "normal" live-load event and a "55-mph" wind event with load factors of 1.35 and 0.4, respectively.

For components not traditionally governed by wind force effects, the Strength III and Strength V load combinations do not govern. Generally, the Strength I load combination governs for a typical multi-girder highway overpass.

Service Load Combinations

Unlike the Strength load combinations, the Service load combinations are material dependent.

Extreme-Event Load Combinations

The Extreme-Event limit states differ from the Strength limit states, because the event for which the bridge and its components are designed has a greater return period than the 75-year design life of the bridge (or a much lower frequency of occurrence than the loads of the Strength limit state).

Fatigue-and-Fracture Load Combination

The Fatigue-and-Fracture load combination, although strictly applicable to all types of superstructures, only affects the steel elements, components, and connections of a limited number of steel superstructures. Chapter 15 discusses fatigue and fracture for steel.

Application of Multiple-Valued Load Factors

Maximum and Minimum Permanent-Load Load Factors. In LRFD Table 3.4.1-1, the variable γ_P represents load factors for all of the permanent loads, shown in the first column of load factors. This variable reflects that the Strength and Extreme-Event limit state load factors for the various permanent loads are not single constants, but they can have one of two extreme values. LRFD Table 3.4.1-2 provides these two extreme values for the various permanent load factors, maximum, and minimum. These maximum and minimum values do not represent a usable range of values. Either the maximum or the minimum value shall apply, not both. Further, in a single load-combination evaluation, the bridge engineer applies either the maximum or the minimum value uniformly to the permanent load, not a combination of the two values. Permanent loads are always present on the bridge, but the nature of uncertainty is that the actual loads may be more or less than the nominal specified design values. Therefore, maximum and minimum load factors reflect this uncertainty.

Select the appropriate maximum or minimum permanent-load load factors to produce the more critical load effect. For example, in continuous superstructures with relatively short-end spans, transient live load in the end span causes the bearing to be more compressed, while transient live load in the second span causes the bearing to be less compressed and perhaps lift up. To check the maximum compression force in the bearing, place the live load in the end span and use the maximum DC load factor of 1.25 for all spans. To check possible uplift of the bearing, place the live load in the second span and use the minimum DC load factor of 0.90 for all spans.

Superstructure design uses the maximum permanent-load load factors almost exclusively; the most common exception is uplift of a bearing as discussed above.

With the use of maximum and minimum load factors, the *LRFD Specifications* have generalized load situations such as uplift where a permanent load (in this case a dead load) reduces the overall force effect (in this case a reaction). Select permanent load factors, either maximum or minimum, for each load combination to produce extreme force effects.

Substructure design routinely uses the maximum and minimum permanent-load load factors from LRFD Table 3.4.1-2. An illustrative yet simple example is a spread footing supporting a cantilever retaining wall. When checking bearing, factor up the weight of the soil (EV) over the heel by the maximum load factor, 1.35, because greater EV increases the bearing pressure, q_{ult}, making the limit state more critical. When checking sliding, factor EV by the minimum load factor, 1.00, because lesser EV decreases the resistance to sliding, Q_{τ}, again making the limit state more critical. Foundation and substructure design requires the application of these maximum and minimum load factors.

12.2. Permanent Loads

12.2.1. General

Reference: LRFD Article 3.5

The *LRFD Specifications* specify seven components of permanent loads, which either are direct gravity loads or caused by gravity loads.

Consider the primary forces from prestressing to be part of the resistance of a component. Omit these from the list of permanent loads in Section 3 of the *LRFD Specifications*. However, when designing anchorages for prestressing tendons, the prestressing force is the only load effect, and appears on the load side of the LRFD Equation. The permanent load EL includes secondary forces from pre-tensioning or post-tensioning. As specified in LRFD Table 3.4.1-2, use a constant load factor of 1.0 for both maximum and minimum load factors for EL.

12.2.2. Superstructure Gravity Loads (DC and DW)

Include a uniform load of 50 psf to account for a wearing surface over the entire deck area between the face of rails or sidewalks. Although not normally permitted in new designs, where steel stay-in-place formwork is used, account for the steel form weight and any additional concrete in the flutes of the formwork.

12.2.3. Distribution of Gravity Loads to Girders

Reference: LRFD Article 4.6.2.2.1

Superimposed dead loads (e.g., curbs, barriers, sidewalks, parapets, railings, wearing surfaces) may be distributed equally to all girders as traditionally specified by AASHTO. For wider bridges with more than six girders, assume that the superimposed dead loads of sidewalks, parapets, or railings are carried by the three girders immediately under and adjacent to the load. In some cases, such as staged construction and heavier utilities, special consideration may be required.

12.2.4. Downdrag on Deep Foundations (DD)

Reference: LRFD Article 3.11

Deep foundations through unconsolidated soil layers may be subject to downdrag. Downdrag is the phenomenon of a soil moving relative to a deep-foundation element in the same direction as the load applied to the element, typically due to consolidation of soft soils underneath embankments. Drag load, also referred to as downdrag load, is the load developed along the vertical sides of the foundation element due to downdrag.

Calculate this additional load as a skin-friction effect. If possible, detail the deep foundation to mitigate the effects of downdrag; otherwise, it is necessary to design considering downdrag. Chapter 17 discusses mitigation methods.

12.2.5. Differential Settlement (SE)

Differential settlement between adjacent substructure units or transversely across a single substructure unit induces stresses in continuous structures and deflections in simple structures. Although most bridges can easily resist these stresses and deflections, consider the potential effects of differential settlement where applicable.

12.3. Transient Loads

12.3.1. General

The *LRFD Specifications* recognize 19 transient loads, which integrate static water pressure, stream pressure, buoyancy, and wave action as water load, WA. The *LRFD Specifications* elevate creep, settlement, shrinkage, and temperature (CR, SE, SH, TU, and TG) in importance to "loads," being superimposed deformations which, if restrained, will result in force effects. For example, restrained strains due to increasing uniform temperature induce compression forces.

12.3.2. Vehicular Live Load (LL)

General

Reference: LRFD Articles 3.6.1.1, 3.6.1.2, and 3.6.1.3

For short and medium span bridges, which predominate in Alaska, vehicular live load is the most significant component of load. Live load becomes less significant for long-span bridges. Long-span bridges are defined as those governed by the Strength IV load combination where the dead load is seven times or more greater than the live load.

The Nature of the Notional Load

The HL-93 live-load model is a notional load in that it is not a true representation of actual truck weights. Instead, the force effects (i.e., moments, shears) due to the superposition of vehicular and lane load within a single design lane are a true representation of the force effects due to actual trucks.

The components of the HL-93 notional load are:

- a vehicle, either the design truck (similar to the former HS20 truck), or a 50-kip design tandem; and
- a 0.64 k/ft uniformly distributed lane load, similar to the lane load of the *Standard Specifications*, but without any of the previous associated concentrated loads.

A dynamic load allowance (IM) of 0.33 is applicable only to the design truck and the design tandem, but not to the uniformly distributed lane load.

The force effects of the design truck alone are less than that of current legal highway loads. Thus, a heavier vehicle is appropriate for design. As specified for the HL-93 live-load model, the concept of superimposing the design vehicle force effects and the design lane force effects was developed to yield moments and shears representative of real trucks on the highways.

Multiple Presence Factors

The multiple presence factor of 1.0 for two loaded lanes, as given in LRFD Table 3.6.1.1.2-1, is the result of the *LRFD Specifications*' calibration for the notional load, which has been normalized relative to the occurrence of two side-by-side, fully correlated, or identical, vehicles. Use the multiple presence factor of 1.2 for one loaded lane where a single design tandem or single design truck governs (e.g., overhangs, decks) or for single-lane bridges. Do not apply the multiple-presence factors to fatigue loads.

Load Applications

Reference: LRFD Article 3.6.1.3.1

General. Neglect axles that do not contribute to the extreme force effect under consideration (e.g., continuous girders).

Two Design Trucks in a Single Lane for Negative Moment and Interior Reactions

Reference: LRFD Article 3.6.1.3.1

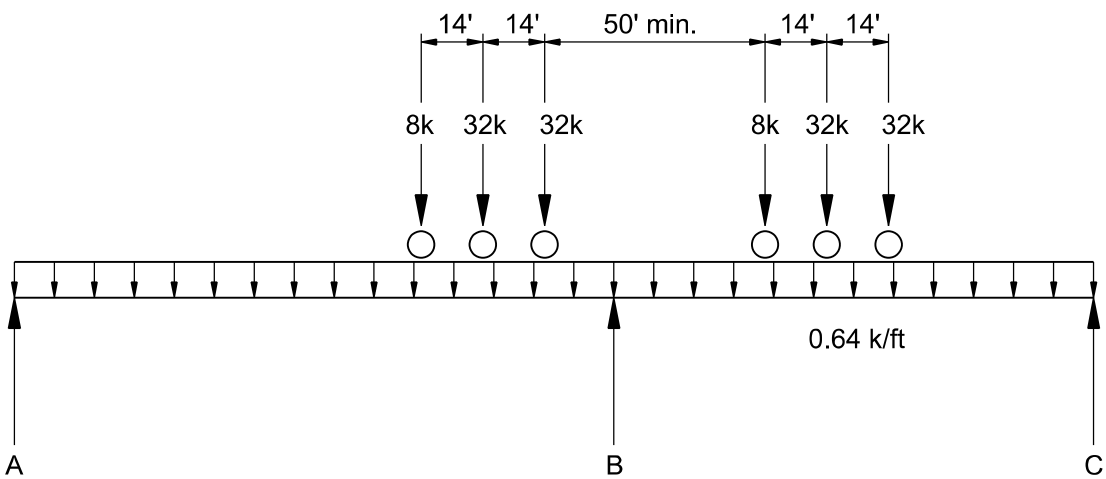
The combination of the lane load and a single vehicle (either a design truck or a design tandem) does not always adequately represent the real-life loading of two heavy vehicles closely following one another, interspersed with other lighter traffic. Thus, the *LRFD Specifications* specify a special load case to calculate these force effects. Two design trucks, with a fixed rear axle spacing of 14 feet and a clear distance not less than 50 feet between them, superimposed upon the lane load, all within a single design lane and adjusted by a factor of 0.90 approximates a statistically valid representation of negative moment and interior reactions due to closely spaced heavy trucks. The *LRFD Specifications* specify this sequence of highway loading for negative moment and reactions at interior piers due to the shape of the influence lines for such force effects. The *LRFD Specifications* do not extend this sequence to other structures or portions of structures because it is not expected to govern for other influence-line shapes. Figure 12-1 illustrates this loading.

In positioning the two trucks to calculate negative moment or the interior reaction over an internal support of a continuous girder, spans should be at least 90 feet in length to be able to position a truck in each span's governing position (over the peak of the influence line). If the spans are larger than 90 feet in length, the trucks remain in the governing positions but, if they are smaller than 90 feet, the bridge engineer can attain the maximum force effect by trial-and-error with either one or both trucks in off-positions (i.e., non-governing positions for each individual span away from the peak of the influence line). When using software, the clear distance between the design trucks will likely need to be varied to determine the maximum force effect. See Figure 12-2.

Application of Horizontal Superstructure Forces to the Substructure. The transfer of horizontal superstructure forces to the substructure depends on the type of superstructure to substructure connection. Assume centrifugal force (CE), braking force (BR), and wind on live load (WL) act horizontally at a distance of 6 feet above the roadway. Connections can be fixed, pinned, or free for both moment and shear.

If the horizontal superstructure force is applied to the substructure through a pinned connection, there is no moment transfer. Apply the superstructure force to the substructure at the connection.

For a fixed or moment connection, apply the superstructure horizontal force with an additional moment to the substructure as shown in Figure 12-3. The additional moment is equal to the horizontal force times the distance between the force's line of action and the point of application.



Note: *Under special loading, use 90% of above.*

Figure 12-1

Special Loading for Negative Moment and Interior Reactions of Continuous Spans

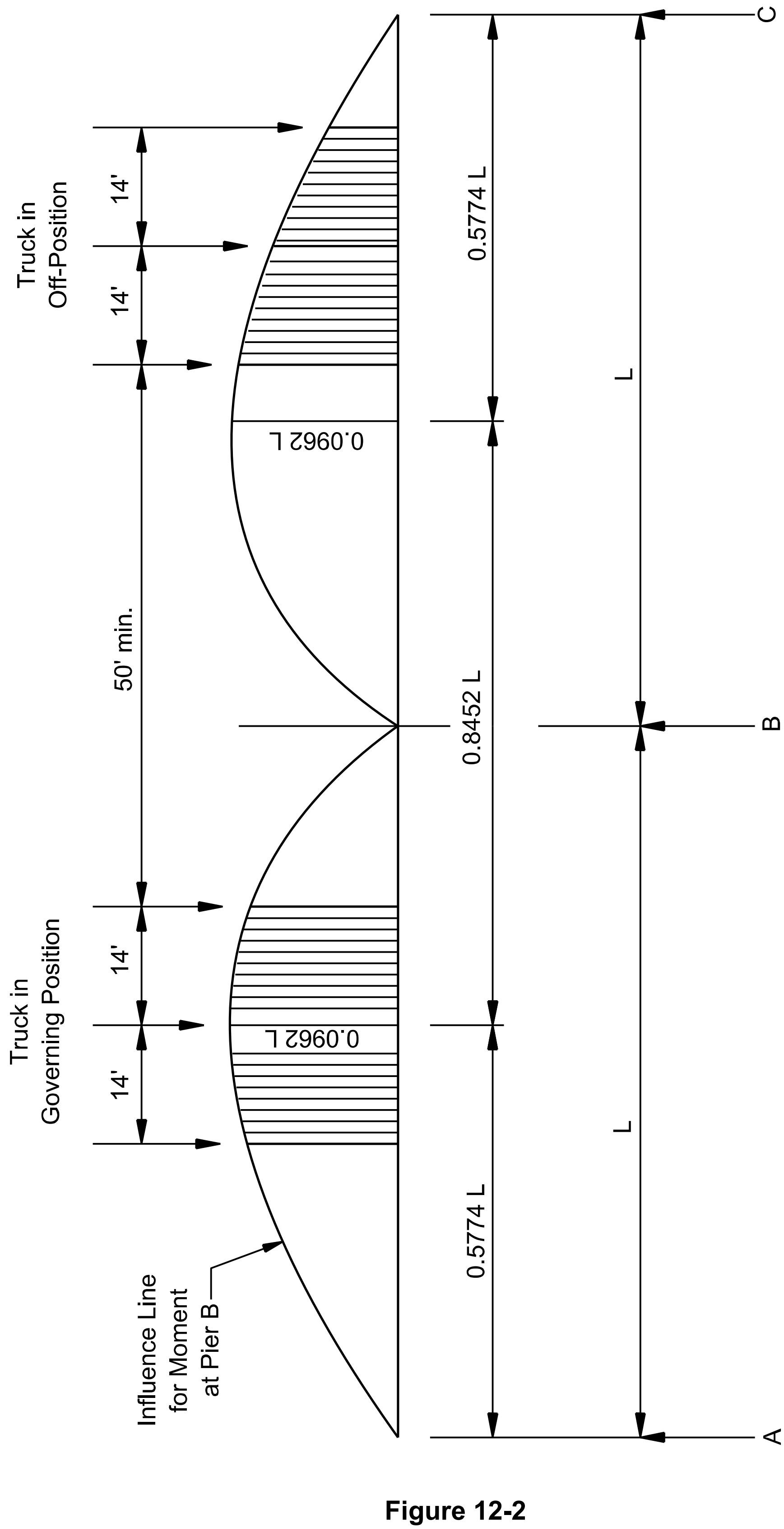


Figure 12-2
Application of Design Vehicular Live Load – LRFD Article 3.6.1.3

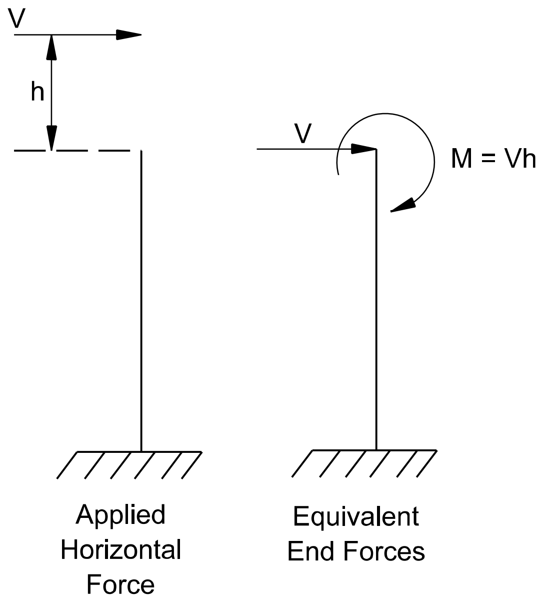


Figure 12-3

Transfer of Horizontal Superstructure Force to Substructure Through Moment Connection

Wheel Load for Deck Design

Reference: LRFD Article 3.6.1.3.3

Design bridge decks to carry axles consisting of two 16-kip wheels with dynamic allowance, alone or in combination with the lane load as appropriate. The design tandem need not be used for the design of decks.

Localized Vehicles

Investigate localized heavy vehicles such as oil field hauling equipment (B-train or oil-field equipment). If localized heavy vehicles are present, consider a site-specific live-load model with the approval of the Chief Bridge Engineer.

Fatigue Loads

Reference: LRFD Articles 3.6.1.4.1, 3.6.1.4.2

The *LRFD Specifications* define the fatigue load for a particular bridge component by specifying both a magnitude and a frequency. The Fatigue I load combination is associated with infinite life, but the Fatigue II load combination is associated with the number of cycles for a 75-year life.

Distribution of Live Load to Piers

Reference: LRFD Article 3.6.1.3.1

To promote uniformity of distribution of live load to piers and other substructure components, use the following procedure unless a more exact distribution of loads is used:

1. **Live-Load Distribution Factor**. Determine the live-load distribution factor for each girder assuming that the deck is acting as a simple beam between interior girders and as a cantilever spanning from the first interior girder over the exterior girder (Lever Rule).
2. **Live Load on Design Lanes**. Place design lanes on the bridge to produce the maximum force effect for the component under investigation. Place the HL-93 live load within its individual design lane to likewise produce the maximum effect. Consider one, two, three, or more design lanes in conjunction with the multiple presence factors of LRFD Table 3.6.1.1.2-1, as can be accommodated on the roadway width.
3. **Reaction on Piers**. For continuous girders or multiple simple span girders, use 90 percent of two closely spaced (i.e. 50 feet) design trucks superimposed over the lane load, with a

distribution factor derived as discussed above in a line-girder analysis to determine the reaction on piers. This is as specified in LRFD Article 3.6.1.3 for negative moment in continuous girders and interior reactions and discussed in Section 12.3.2.4.2.

Sidewalk Loading
Reference: LRFD Article 3.6.1.6

Where sidewalks are present on the bridge, design for the dead load and pedestrian live load on the sidewalk; however, also design the full width of the bridge, including sidewalks, for the traffic live load assuming that traffic can mount the sidewalk. Do not apply pedestrian and traffic loads concurrently. Design for vehicular loads any sidewalks separated from traffic lanes by barrier rail to account for maintenance vehicles and potential future widening.

12.3.3. Friction Forces (FR)

Reference: LRFD Article 3.13

Adjust the frictional forces from sliding bearings to account for unintended additional friction forces due to the future degradation of the coefficient of friction of the sliding surfaces. Consider the horizontal force due to friction conservatively. Include friction forces where design loads would increase, but neglect friction forces where design loads would decrease.

12.3.4. Thermal Loads

Reference: LRFD Article 3.12.2

Use a modified Procedure A of LRFD Article 3.12.2.1 to determine the appropriate design thermal movement range. For Alaska-specific ranges of temperatures and procedures, see Chapter 19 on the design of joints and bearings.

12.3.5. Earthquake Effects (EQ)

Reference: *Guide Specifications for LRFD Seismic Bridge Design*

Use the AASHTO *Guide Specifications for LRFD Seismic Bridge Design* to design bridges in Alaska. Other chapters in this *Manual* present DOT&PF's seismic detailing practices.

12.3.6. Live-Load Surcharge (LS)

Reference: LRFD Article 3.11.6.4

Where approach slabs are provided at bridge ends, consider the reactions on the abutment and wingwall due to the axle loads on the approach slabs plus one-half of the live-load surcharges specified in LRFD Article 3.11.6.4. This applies to walls parallel to or perpendicular to the roadway centerline.

Retaining walls that retain soil supporting a roadway must be able to resist the lateral pressure due to the live-load surcharge. See Chapter 21 for retaining walls.

12.3.7. Vessel/Collision (CV)

Reference: *Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges*, 2nd Edition

Vessel collision is a site-specific consideration that the bridge engineer will consider on a case-by-case basis in active boating channels.

12.3.8. Ice Loads

Reference: LRFD Article 3.9

Apply ice loads as specified in LRFD Article 3.9, but be aware of special situations where historical ice loads have occurred.

Consider ice loads in the conceptual design of the bridge. For example, where historical ice loads have caused problems, consider whether to place a pier in the water.

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13. Structural Analysis and Evaluation

13.1. Distribution of Live Load

Reference: LRFD Article 4.6.3.1

13.1.1. Definition

Live-load distribution, for application to the *Alaska Bridges and Structures Manual*, refers to determining the maximum number of loaded lanes that an individual girder of the superstructure will be expected to carry. The live-load distribution factor is the maximum number of loaded lanes per girder.

13.1.2. Approximate Methods

Reference: LRFD Article 4.6.2

General

Distribution factors allow for a simple, approximate analysis of bridge superstructures. Live-load distribution factors uncouple the transverse and longitudinal distribution of force effects in the superstructure. The approximate method distributes live-load force effects transversely by proportioning the design lanes to individual girders through the application of distribution factors. Subsequently, this method distributes the force effects longitudinally between the supports through the one-dimensional (1-D), line-girder structural analysis over the length of the girders.

Use distribution factors and 1-D, line-girder analysis where allowed by the *LRFD Specifications*. Distribution factors reduce the necessity of modeling the entire bridge using a 2-D or 3-D analysis.

Simplified Analysis

Reference: LRFD Article 4.6.2.2

General. LRFD Article 4.6.2.2.2 presents several common bridge superstructure types, with empirically derived equations for live-load distribution factors for each type. Each distribution factor provides a number of design lanes to be applied to a girder to evaluate the girder for moment or shear. The factors account for interaction among loads from multiple lanes and the effects of skewed supports.

The distribution factors represent the placement of design lanes to generate the extreme effect in a specific girder. The location of design lanes is unrelated to the location of striped traffic lanes on the bridge.

The properties used in calculating the live-load distribution factors vary along the span; for example, steel plate girder moments of inertia vary at the flange or web plate transitions. However, do not recalculate the distribution factor at each change in property. Use weighted average properties or maximum properties (e.g., in the span for positive moment and at the pier for negative moment) to calculate single acceptable distribution factors.

Limitations. The Chief Bridge Engineer must approve using the distribution-factor equations beyond the “Range of Applicability” without the use of a refined analysis. See Section 13.2 for a discussion on refined analyses.

Skewed Bridges. Do not use the skew correction factors for moment in LRFD Table 4.6.2.2.2e-1 to adjust the live load moments in skewed bridges.

Torsional moments exist about the longitudinal axis in skewed bridges due to gravity loads (both dead and live load). These moments increase the reactions and shear forces at the obtuse corners compared to the acute corners. The potential exists for reactions to become very small or negative at acute corners; avoid this condition when possible. The bridge engineer should account for the higher reactions at the obtuse corners in the design of bearings and the supporting elements.

Use the skew correction factors for shear in LRFD Table 4.6.2.2.3c-1 to adjust the live load shears and reactions in skewed bridges. Decrease the skew correction factor for shear linearly from the centerline of bearing to 1.0 at midspan. Use the following skew correction factor for decked bulb-tee girders:

$$Skew\ factor = 1.0 + 0.20\left(\frac{12.0Lt_s{}^3}{K_g}\right)^{0.3} tan\theta$$

where:

L = span length (ft.)
t_s = top flange thickness (in.)
K_g = I_x for monolithic sections (in.^{4})
θ = skew angle (degrees)

Historically, top flange thicknesses between 5 and 7 inches have been used. Newer designs use 6-inch thick top flanges on paved bridge and 7-inch thick top flanges on unpaved bridges.

Decked Bulb-Tee Girder Example

The next two pages present a girder distribution of live loads for moment example for a decked bulb-tee girder bridge. Distribution of live loads for shear requires use of the “lever rule,” which is not presented in this example. Use the following format as a template for these types of calculations. For example:

- Provide a cross section of the girder with all dimensions.
- Use the “References/Notes” column to cite the applicable LRFD reference adjacent to each calculation.
- Ensure that a bridge engineer unfamiliar with the project can follow the sequence of calculations.

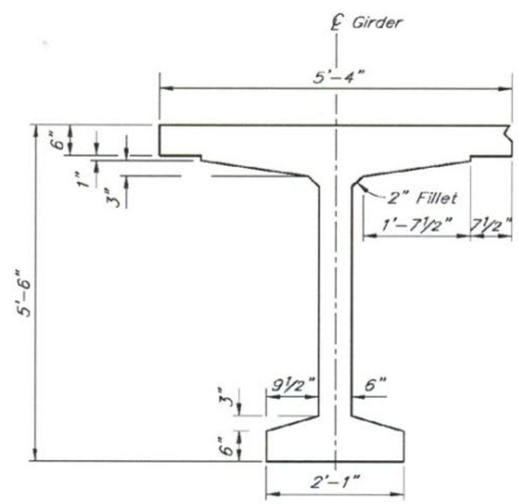
STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

COMPUTATIONS

Bridge No. N/A
Date 8/9/2008
For: **Girder Distribution Example** Calc. By

Reference/Notes

Example: Decked bulb-tee girder bridge with span length, L = 146'-0", deck width, W = 43'-0" including two 1'-6" bridge rails



Section properties for this girder section:

Area, A = 1,004 in^2
Moment of Inertia, I_x = 570,730 in^4
I_y = 160,100 in^4

Tosional Constant, $J = \dfrac{A^4}{40 * I_p}$

Where, $I_p = I_x + I_y$

And, J = 34,758 in^4

Distribution for Moment in Interior Girders:

$$g_{mi} = \frac{S}{D}$$ AASHTO LRFD Table 4.6.2.2.2b-1

Gap between girders

Where: S = Girder Spacing = 5'-4" + ½" = 5'-4½" or, 5.4 ft

D = First, Calculate:

$C = k * (W/L) < k$

Where: $k = \sqrt{\dfrac{(1+\mu)\cdot I_x}{J}}$

Where: μ = Poisson's Ratio = 0.15 to 0.2 for concrete, will use 0.16

$$k = \sqrt{\frac{(1+0.16)\cdot 570730\,in^4}{34758\,in}}$$

or, k = 4.4

W = Deck Width = 43 ft
L = Span Length = 146 ft

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AND
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COMPUTATIONS

Bridge No. N/A
Date 8/9/2008
For: **Girder Distribution Example**
Calc. By

And, C = 4.4 * (43'/143') = 1.29

Since C < 5,

$D = 11.5 - N_L + 1.4*N_L*(1-0.2*C)^2$

Where, N_L = Number of Lanes = $\frac{43'\text{-}0" - 2*(1'\text{-}6")}{12'\text{-}0" \text{ per lane}}$

= 3.33

or, 3 lanes

$D = 11.5 - 3 + 1.4*3*(1-0.2*1.29)^2$

= 10.8 ft

And, $g_{mi} = \frac{5.4 \text{ feet}}{10.8 \text{ feet}}$

= 0.50 lanes/girder

13.2. Refined Analysis

Reference: LRFD Articles 4.6.2.2 and 4.6.3

13.2.1. General

Use a refined analysis only with the approval of the Chief Bridge Engineer and only for bridges where the parameters fall outside of the "Range of Applicability." Where refined analysis is used, show back-calculated live-load distribution factors for each girder in the contract documents for future use in rating or rehabilitating the bridge.

13.2.2. 2-D Analysis (Horizontally Curved Bridges)

Use refined analysis methods, either grid or finite-element, for the analysis of horizontally curved bridges. LRFD Article 4.6.2.2.4 states that approximate analysis methods may be used for the analysis of curved bridges but then highlights the deficiencies of these analyses, specifically the V-load method for I-girders and the M/R method for boxes. Therefore, DOT&PF does not allow the sole use of these methods for horizontally curved bridges. Use the V-load method for preliminary design purposes or as an order-of-magnitude checking tool.

Table 13-1 provides an example DF table for plans:

Table 13-1
Design Live Load Distribution Factors

Girder Designation	Force Effect	Multiple Loaded Lanes	Single Loaded Lane
Exterior	+ Moment (near midspan)	0.88	0.69
	- Moment (at piers)	0.81	0.60
	Shear (near supports)	1.04	0.87
Interior	+ Moment (near midspan)	0.65	0.37
	- Moment (at piers)	0.74	0.41
	Shear (near supports)	0.74	0.56

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14. Structural Concrete

Section 5 of the *LRFD Specifications* presents unified design requirements for concrete, both reinforced and prestressed, in all structural elements. This chapter presents DOT&PF supplementary information specifically on the properties of concrete, reinforcing steel, and prestressing strands and the design of structural concrete members.

14.1. Materials

14.1.1. Structural Concrete

Reference: LRFD Article 5.4.2

Table 14-1 presents DOT&PF criteria for the minimum 28-day compressive strength of concrete in structural elements. Normal weight concrete varies between 145 pcf (for cast-in-place concrete) and 155 pcf (for precast concrete) excluding the weight of the internal steel reinforcement.

14.1.2. Reinforcing Steel

Reference: LRFD Article 5.4.3

For both general and seismic applications, reinforcing steel must conform to the requirements of ASTM A706, Grade 60. (A706 is available as either Grade 60 or Grade 80, but Grade 80 is not permitted in members expected to form plastic hinges such as columns.) Use 29,000 ksi as the modulus of elasticity, E_s. ASTM A706 reinforcing steel is manufactured with controlled material properties. These properties include a maximum yield strength, a minimum ratio between the tensile and yield strengths, and a higher rupture strain than conventional ASTM A615 reinforcement. In addition, ASTM A706 reinforcing steel is manufactured with a controlled chemical composition making it more weldable. Comply with *Alaska Standard Specifications* Section 503 for welding of reinforced steel.

The bridge engineer may substitute ASTM A615 only with the approval of the Chief Bridge Engineer.

Seismic Modeling of Reinforcing Steel

When designing reinforced concrete members that are expected to form plastic hinges (i.e., earthquake-resisting elements) according to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* use the steel reinforcement properties in Table 14-2. The code provisions are based primarily on preserving life safety through a "no collapse" design objective. Some bridges may be required to perform to a higher design objective following a seismic event.

Bridges that suffer minimal damage may be expected to open shortly after the design seismic event provided that the strain in the concrete compression face not exceed 0.004 and the strain in the extreme longitudinal tension reinforcing does not exceed the onset of strain hardening, ε_{sh}, as shown in Table 14-2.

Circular column damage following the design seismic event is expected to be repairable provided that the longitudinal reinforcing steel does not buckle (Goodnight et. al. 2015). For circular columns satisfying $\rho_l < 0.03$ and $\rho_s > 0.005$, the longitudinal steel strain associated with the onset of bar buckling, ε_s^{bb}, may be calculated as:

$$\varepsilon_s^{bb} = 0.032 + 790\rho_s F_{yhe}/E_s - 0.14P / (f'_{ce}A_g)$$

where:

A_g = gross area of member cross section (in.^{2})

A_{sp} = area of spiral or hoop reinforcing bar (in.^{2})

A_{st} = total area of column reinforcement (in.^{2})

A_{st} = total area of column reinforcement (in.^{2})

D' = core diameter of column measured from center of spiral or hoop (in.)

E_s = modulus of elasticity of reinforcing steel (ksi)

f'_{ce} = expected concrete compressive strength (ksi)

F_{yhe} = expected strain of transverse reinforcing steel (ksi)

P = unfactored axial dead load on column (k)

S = pitch of spiral or spacing of hoop reinforcement (in.)

ρ_l = longitudinal reinforcement ration in the column

= A_{st} / A_g

ρ_s = volumetric reinforcement ratio

= $4*A_{sp} / (s * D')$

The repairable strain limits for non-circular column cross sections have not been developed.

Table 14-1
Compressive Strength of Concrete

Structural Element	Minimum 28-Day Compressive Strength (f'_c)
Bridge Decks (Class AA Concrete)	5 ksi
Prestressed Concrete (Precast) (Class P Concrete)	5 ksi
Prestressed Concrete (Cast-in-Place) (Class A Concrete)	4 ksi
Piers and Columns (Class A Concrete)	4 ksi
Abutments (Class A Concrete)	4 ksi
Wingwalls and Retaining Walls (Class A Concrete)	4 ksi
Spread Footings (Class A Concrete)	4 ksi
Drilled Shafts (Class DS Concrete)	4 ksi
Barrier Rails and Rail Curb, etc. (Class A Concrete)	4 ksi
Miscellaneous (e.g., Culverts, Sound Walls) (Class A Concrete)	4 ksi
Approach Slabs	See 16.3

14.1.3. Prestressing Strands

Reference: LRFD Article 5.4.4

Use low-relaxation, 7-wire prestressing strand with a minimum tensile strength of f_{pu} = 270 ksi and a minimum yield strength of $f_{py} = 0.9f_{pu}$. If more precise data is not available, the modulus of elasticity for prestressing strands, based on nominal cross-sectional area, may be taken as 28,500 ksi.

Table 14-2
Stress Properties of Reinforcing Steel Bars

Property	Notation	Bar Size	ASTM A706 Grade 60	ASTM A706 Grade 80
Specified minimum yield stress (ksi)	f_y	#3-#18	60	80
Expected yield stress (ksi)	f_{ye}	#3-#18	68	85
Expected tensile strength (ksi)	f_{ue}	#3-#18	95	112
Expected yield strain	ε_{ye}	#3-#18	0.0023	0.0033
Onset of strain hardening	ε_{sh}	#3-#8	0.0150	0.0074
		#9	0.0125	0.0074
		#10 & #11	0.0115	0.0074
		#14	0.0075	0.0074
		#18	0.0050	0.0074
Reduced ultimate tensile strain	ε_{su}^{R}	#4-#10	0.090	0.06
		#11-#18	0.060	0.06
Ultimate tensile strain	ε_{su}	#4-#10	0.120	0.095
		#11-#18	0.090	0.095

14.2. Reinforcement

14.2.1. Reinforcing Steel

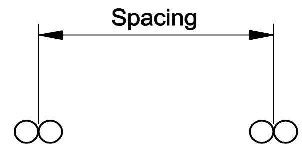
Spacing of Bars

Reference: LRFD Article 5.10.3

Table 14-3 presents DOT&PF criteria for minimum center-to-center spacing between reinforcement bars based on bar size and spliced vs unspliced. The accompanying sketch illustrates how to measure the spacing for spliced bars. Epoxy-coated bars spaced at less than six bar diameters, which includes the majority of the table, require an additional increase in development lengths over wider spaced epoxy-coated bars.

Use calculations and large-scale drawings to carefully check fit and clearance of reinforcing. Skews will tend to complicate problems with reinforcing fit. Consider tolerances normally allowed for cutting, bending, and locating reinforcing. Refer to the *Alaska Standard Specifications for Highway Construction* for tolerances.

Table 14-3
Minimum Spacing of Bars

Bar Size	Minimum Spacing: Unspliced Bars	Minimum Spacing: Spliced Bars (assumes a side-by-side lap) 
#4	3″	3½″
#5	3″	3½″
#6	3″	4″
#7	3½″	4″
#8	3½″	4½″
#9	3½″	4½″
#10	4″	5″
#11	4″	5½″
#14	4½″	6″
#18	6″	8″

Fabrication Lengths

Use a maximum unspliced length of 60 feet for detailing reinforcing steel. In remote locations, use 40 feet (for shipping).

Lateral Confinement Reinforcement

Reference: LRFD Article 5.11.4.

Detail all lateral column reinforcement according to the requirements in the *Guide Specifications for LRFD Seismic Bridge Design*. Lateral reinforcement for compression members must consist of either spiral reinforcement or electric resistance butt-welded hoops.

Lateral Confinement Reinforcement for Drilled Shafts. The reinforcing steel cage for drilled shafts must extend the full depth of the shaft.

Determine the length of the plastic hinge confinement reinforcement by appropriate analysis, but the length must not be less than the requirements of the *Guide Specifications for LRFD Seismic Bridge Design*.

Maximize the size of longitudinal and transverse reinforcement to increase the openings between all reinforcement to allow concrete to pass through the cage during placement. The window formed between the longitudinal and transverse bars should not be less than 5 inches by 5 inches. Maintain the maximum spacing requirements of LRFD Article 5.11.4.5.2.

Epoxy-Coated Reinforcing Steel

Epoxy-coated reinforcement bars are required at the following locations:

- bridge decks (both layers)

- reinforcing that extends into bridge decks and terminates within 12 inches of the top of the deck slab
- bridge approach slabs
- barrier rails and rail curbs
- sidewalks

Standard End Hook Development Length in Tension
Reference: LRFD Article 5.10.8.2.4.

Closed ties always fold the hooked end at 135° into the core. See Figure 14-1. Make both tails parallel for confinement with the specified length (LRFD Article 5.10.2.2). Standard hooks use a 90-degree and 180-degree bend to develop bars in tension where space limitations restrict the use of straight bars. End hooks on compression bars are not effective for development length purposes.

For the development of standard hooks, refer to Figure C5.10.8.2.4a-1 in the *LRFD Specifications* for hooked-bar details. Use the same figure for both uncoated and coated bars.

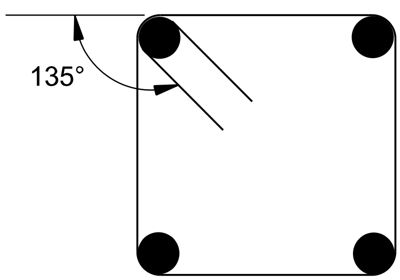


Figure 14-1
End Hook for Closed Ties

Splices
Reference: LRFD Article 5.10.8.4.

1. **Deck Steel over Piers.** Over piers, do not provide any splices in the longitudinal deck reinforcing steel where the tensile stress in the deck is calculated to be greater than the rupture modulus of the concrete. The contract documents must clearly identify these regions. In practice, especially in remote locations, splices may be unavoidable. In these cases, optimize the location of the no-splice zone and indicate it on the plans.

2. **Plastic Hinge Regions.** In columns and drilled shafts, do not provide any splices in the longitudinal reinforcing or splicing of spiral reinforcing within the plastic hinge regions. The contract documents must clearly identify these regions. In practice, especially in remote locations, splices may not be avoidable. In these cases, optimize the location of the no-splice zone and stagger the splices as much as practical.

Spiral Reinforcing Steel Calculation
Use the equation $S = n\,((\pi d)^2 + h^2)^{1/2}$ to calculate the length of spiral reinforcing steel for any spiral segment with a constant pitch, where:

h = pitch of spiral segment
n = number of turns
d = outer bar diameter

To calculate the total length of reinforcing steel in a spiral, add the length of each segment, S, with the length of additional top and bottom turns.

14.2.2. Prestressing Strands for Pretensioned Girders

Strand Size
Common sizes of prestressing strands used in bridge construction are ½-inch and 0.6-inch diameter. However, for girders fabricated within Alaska, the diameter of the prestressing strands in pretensioned girders is ½ inch. For girders fabricated outside of Alaska, the diameter is typically 0.6 inch.

Strand Spacing
The typical strand spacing is 2 inches center-to-center.

Strand Profile
It is acceptable to use either a straight or harped strand profile for precast members. DOT&PF prefers the use of harped strands over the debonding of straight strands. Use a combination of debonded and harped strands when necessary to satisfy design requirements, subject to the following:

- Only use debonded strands when required by analysis.
- Do not debond more than 40 percent of the strands.
- Do not debond more than one-half of the strands in a row.
- Do not debond harped strands.

Harped Strands

In precast, pretensioned girders harped strands are bundled at the harp points and the slope of the harped strands should not exceed 1:7 (9 degrees).

Strand Patterns

Fully detail the strand pattern showing the total number of strands, layout and spacing, edge clearances, which strands will be harped and/or debonded, and the layout of all mild reinforcing steel.

Frequently, it may be preferable to design precast, pretensioned girders of the same size and similar length in the same bridge or within bridges on the same project with a slightly different number of strands. In this case, consider using the same number and pattern of strands (including height of harping) for these girders to facilitate fabrication. Figure 14-2 presents a typical strand pattern.

Strand Splicing

Do not splice prestressing strand.

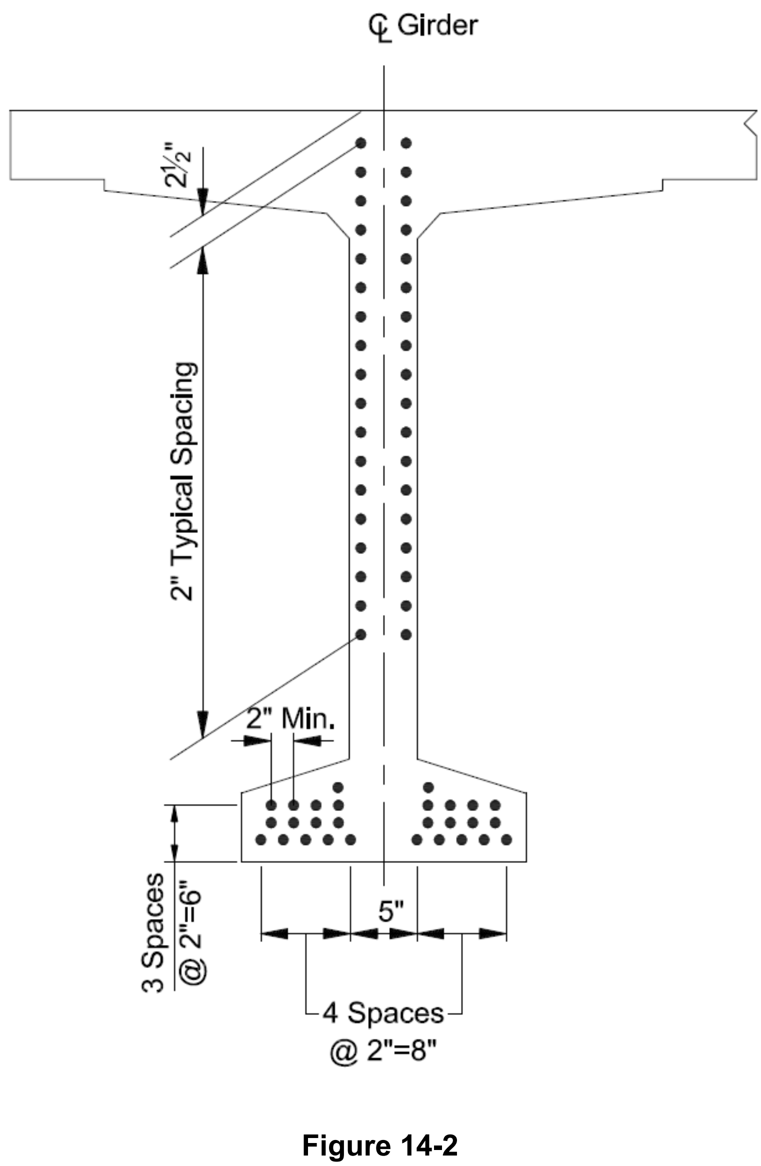


Figure 14-2
Typical Strand Pattern

14.3. Structural Concrete Design

14.3.1. Member Design Models

The *LRFD Specifications* provides two design approaches for concrete members — the traditional sectional design model and the strut-and-tie model. The sectional design model is based upon traditional beam theory wherein planar sections remain plane after loading. The strut-and-tie model may be used where traditional theory is applicable and in disturbed regions where planar sections do not remain planar after loading.

14.3.2. Sectional Design Model

The sectional design model is appropriate for the design of typical bridge girders, slabs, columns, and other regions of components where the assumptions of traditional beam theory are valid. The sectional design model assumes that the response at a particular section depends only on the calculated values of the sectional force effects such as moment, shear, axial load, and torsion. This model does not consider the specific details of how the force effects were introduced into the member. LRFD Article 5.7.3. discusses the sectional design model and describes the applicable geometry required to use this technique to design for shear.

Flexural Resistance

Obtain the flexural resistance of a girder section by using the rectangular stress distribution of LRFD Article 5.6.2.2. In lieu of using this simplified, yet accurate approach, use a strain compatibility approach as outlined in LRFD Article 5.6.3.2.5. Base the general equation for structural concrete flexural resistance of LRFD Article 5.6.3.2.1. upon the rectangular stress block or other constitutive model capable of accurately representing the magnitude and location of the resultant concrete compressive force.

Minimum flexural reinforcement requirements are specified in LRFD. The LRFD requirements are satisfied when either:

(1) The factored nominal moment, ϕM_n, is greater than 1.33 times the factored ultimate moment, M_u. This requirement does not necessarily result in a flexural member with any significant deformation or ductile behavior. Non-ductile members fail without significant warning, or signs of distress prior to fracture. In this situation, the reserve strength provided by the member is deemed adequate to preclude failure of non-ductile members.

(2) The factored nominal moment, ϕM_n, exceeds the expected concrete cracking moment, M_{cr}, defined by Equation 5.6.3.3-1. Flexural members that satisfy this requirement are capable of developing sufficiently large deformations to provide warning of impending failure. That is, members that satisfy this requirement provide for a ductile response.

Design reinforced concrete flexural members to satisfy the ductility requirements of Equation 5.6.3.3-1.

Crack Control Reinforcement
Reference: LRFD Article 5.6.7.

When designing for crack control, use the following values unless a more severe condition is warranted:

- γ_e = 0.75 (Class 2 exposure condition where the assumed crack width equals 0.013 inch) for footings and other components in contact with soil or brackish water, for decks, slabs, barrier rail, tops of abutment caps below expansion joints, and other components susceptible to deicing agent exposure; and

- γ_e = 1.00 (Class 1 exposure condition where the assumed crack width equals 0.017 inch) for all other components.

Several smaller reinforcing bars at moderate spacing are more effective in controlling cracking (more numerous but narrower crack widths) than fewer larger bars (fewer but wider cracks).

Shear Resistance

Provide adequate shear reinforcement in bridge girders so that the shear and moment load-rating factors are approximately equal (within about 20 percent). The zero-tension stress limit of Section 14.4.2 often results in girders with greater flexural resistance than that required in the Strength limit state load combinations. However, no similar provisions exist for shear. To ensure a ductile bridge response mechanism, additional shear reinforcement will be required.

When calculating the net longitudinal strain, i.e. ε_s or ε_x, include the effects of strand development length, ℓ_d, in the E_pA_{ps} denominator term only. Include the prestress transfer length adjustment in the $A_{ps}f_{po}$ numerator term only.

Sectional Design Models. Sectional design models are appropriate for flexural regions, regions away from reactions, applied loads, and changes in cross section, where conventional methods for the strength of materials are applicable and strains are linear. The LRFD Specifications present two alternative sectional shear design models for estimating the shear resistance of concrete members:

1. **Simplified Procedure for Nonprestressed Sections**. (Reference: LRFD Article 5.7.3.4.1). This procedure may be used for conventionally reinforced concrete members without axial loads that are less than 16 inches. The shear resistance determined by this procedure is essentially identical to those traditionally used for evaluating shear resistance. This procedure can be seriously unconservative for large members not containing transverse reinforcement.
2. **General Procedure: Modified Compression Field Theory (MCFT)**. (Reference: LRFD Article 5.7.3.4.2 and Appendix B5). This is the recommended procedure to determine the shear resistance of concrete members.

14.3.3. Strut-and-Tie Model

Reference: LRFD Article 5.8.2.

Use the strut-and-tie model to determine internal force effects in disturbed regions, regions near reactions, applied loads, or changes in cross section, where the sectional models are not appropriate. Further, it is only applicable to the Strength and Extreme-Event limit states because significant cracking must be present for the model to be valid.

Members, when loaded, indicate the presence of definite stress fields that can individually be represented by tensile or compressive resultant forces as their vector sums. The "load paths" taken by these resultants form a truss-like pattern that is optimum for the given loading and in which the resultants are in reasonable equilibrium, especially after cracking. The objective is to conceive this optimum pattern (truss) in developing the strut-and-tie model. The closer the assumption is to this optimum pattern (truss), the more efficient the use of materials. For relatively poorly conceived strut-and-tie models, the materials will be used less efficiently, yet the structure will be safe. The compressive concrete paths are the struts, and the reinforcing steel groups are the ties. The model does not involve shear or moment because the stresses are modeled as axial loads alone.

The application of the strut-and-tie model is discussed in C5.8.2.1 of *LRFD Specifications.*

The strut-and-tie model has significant application to bridge components such as pier caps, girder ends, post-tensioning anchorage zones, etc. For a thorough presentation of the model, refer to:

- NCHRP 20-7, Task 217 Verification and Implementation of Strut-and-Tie Model in LRFD Bridge Design Specifications, November 2007;
- D. Mitchell, M. Collins, S. Bhidé and B. Rabbat, "*AASHTO LRFD Strut-and-Tie Model Design Examples,*" EB231, Portland Cement Association (PCA);
- Chapter 8 of the PCI Precast Prestressed Concrete Bridge Design Manual; and
- J. Schlaich, et al, "*Towards a Consistent Design of Structural Concrete,*" PCI Journal, Vol. 32, No. 3, 1987.

The *LRFD Specifications* provide for an adequate design; even if the strut-and-tie model is not used for actual proportioning, the model provides a fast check to ensure the adequacy of the design, especially for the appropriate anchorage of the steel.

Concrete cracking is associated with at least partial debonding of reinforcing steel bars; therefore, do not consider the bonding capacity of cracked concrete to be completely reliable. The *LRFD Specifications* generally require that reinforcing steel should not be anchored in cracked zones of concrete. Improperly anchored reinforcing steel (i.e., bars that are not fully developed) is an area that is commonly overlooked. Consider the use of headed reinforcing bars in cracked regions and where the available development length is inadequate to develop the required reinforcement strength.

14.3.4. Torsion

Torsion is not normally a major consideration in most concrete highway bridges. Where torsion effects are present, design the member in accordance with LRFD Articles 5.7.2 and 5.7.3.6. Situations that may require a torsion design include:

- cantilever brackets connected perpendicular to a concrete girder, especially if a diaphragm is not located opposite the bracket;
- concrete diaphragms used to make precast girders continuous for live load where the girders are spaced differently in adjacent spans;
- abutment caps, if they are unsymmetrically loaded; and
- horizontally curved members.

14.4. Prestressed Concrete Girders

14.4.1. General

The generic word "prestressing" relates to a method of construction in which a steel element is tensioned and anchored to the concrete. Upon release of the tensioning force, the concrete will largely be in residual compression and the steel in residual tension.

There are three methods of applying the prestressing force, as discussed below. Only two of these methods, pretensioning and post-tensioning, are acceptable, and a combination of these two methods is acceptable if approved by the Chief Bridge Engineer.

Pretensioning: In the pretensioning method, tensioning of the steel strands is complete before placing the concrete. When the concrete surrounding the steel strands attains a specified minimum strength, the strands are released thereby transmitting the prestressing force to the concrete by bond-and-wedge action at the girder ends. The initial prestress is immediately reduced due to the elastic shortening of the concrete. Further losses will occur over time due to shrinkage and creep of concrete and relaxation of prestressing steel.

The generic word "prestress" is often used to mean "pretensioning" as opposed to "post-tensioning."

Post-Tensioning: In the post-tensioning method, tensioning of the steel is accomplished after the concrete has attained a specified minimum strength. The tendons, usually comprised of numerous strands, are loaded into ducts cast into the concrete. After stressing the tendons to the specified prestressing level, they are anchored to the concrete and the jacks are released.

Several post-tensioning systems and anchorages are used in the United States; the best information may be directly obtained from the manufacturers.

Post-tensioned concrete is also subject to losses from shrinkage and creep, although at a reduced magnitude because a significant portion of shrinkage usually occurs by the time of stressing, and the rate of creep decreases with the age at which the prestress is applied. After anchoring the tendons, the ducts are pressure filled with grout, which protects the tendons against corrosion and provides composite action by bonding the strand and the girder. Post-tensioning can be applied in phases to further increase the load-carrying capacity and better match the phased dead loads being applied to the girder.

Partial Prestressing: In this hybrid design, both mild reinforcement and prestressing are present in the tension zone of a girder.

The idea of partial prestressing, at least to some extent, originated from a number of research projects that indicated fatigue problems in prestressed girders. Fatigue is a function of the stress range in the strands, which may be reduced by placing mild steel parallel to the strands in the cracked tensile zone to share live-load induced stresses. In these projects, based on a traditional model, however, the fatigue load was seriously overestimated.

The fatigue load provided by the *LRFD Specifications* is a single design vehicle with reduced weight that is not likely to cause fatigue problems unless the girder is grossly under-reinforced.

Do not use partial prestressing. Partially prestressed designs usually result in more tension in the girder at Service loads and analytical tools are not readily available to accurately predict stress-strain levels of different steels in the cross section.

14.4.2. Design Criteria for Pretensioned Concrete

This discussion applies to pretensioned concrete members.

Concrete Stress Limits
Reference: LRFD Article 5.9.42.3.

Tensile stress limits for fully prestressed concrete members must conform to the requirements for "Other Than Segmentally Constructed Bridges" in LRFD Article 5.9.42.3., except that the bridge engineer must limit the tensile stress at the Service limit state, after losses, to zero tension.

Use gross-section properties in conjunction with the Service III load combination or transformed-section properties with the Service I load combination. This requirement applies within the transfer length of the prestressing strands in addition to beyond the transfer length.

Concrete Strength at Release
Reference: LRFD Article 5.9.42.3.1.

Calculate the minimum concrete compressive strength at release (f'_{ci}) for each prestressed girder, and show it on the plans. Concrete compressive strengths at release of between 5.0 ksi and 8.0 ksi are typical. For specified concrete release strengths less than 7 ksi, round up the value shown on the plans to the next increment of 0.25 ksi. For release strengths greater than 7 ksi, round up to the next increment of 0.1 ksi.

Prestressing-Strand Stress Limits
Reference: LRFD Article 5.9.2.2.

Limit prestressing-strand stress immediately prior to transfer to 70 percent of f_{pu}.

Loss of Prestress
Reference: LRFD Article 5.9.3.

The loss of prestress is the difference between the initial stress in the strands and the effective prestress in the member. This includes both instantaneous and time-dependent losses.

Only use low relaxation strand. Estimate the time-dependent losses for decked bulb-tee girders with the following equation:

$$\Delta f_{pLT} = 33\left[1 - 0.15\left(\frac{f_c' - 6}{6}\right)\right] - 2$$

Maximum Stirrup Spacing
Do not exceed a stirrup spacing of 18 inches.

Haunch Thickness for Design
For girders supporting a cast-in-place concrete slab, consider the haunch to have maximum thickness for dead-load calculations and to be non-existent for girder-resistance calculations.

14.4.3. Precast Girder Sections

Reference: LRFD Article 5.9

This section addresses the general design theory and procedure for precast, prestressed (pretensioned) concrete girders. For additional design examples, consult the *PCI Bridge Design Manual*, Chapter 9.

Standard Girder Section
There is currently only one precast girder manufacturer in Alaska. This fabricator can produce a limited number of girder styles and sizes. Contact girder manufacturers when a desired girder shape is not readily available within the state.

1. **Precast Decked Bulb-Tee Sections.** Figure 14-3 shows the standard Alaska style precast decked bulb-tee section. The figure shows a 66-inch deep section. Fabricators create other sections by varying the member depth from 42 inches through 66 inches in 12-inch increments. Thirty six-inch deep members may also be available, depending on the number of girders required. Thirty-five-inch deep members may be available but are rarely used. Decked bulb-tee girder widths range from 49 inches up to 8½ feet but are typically between 5½ feet and 7½ feet. Table 14-4 presents the required deck reinforcing. Figure 14-4 presents the location and shape of typical bulb-tee reinforcing steel.

2. **Precast Bulb-Tee Sections.** Precast bulb-tees are available without the monolithic deck. These sections, intended to support a separate reinforced concrete deck, are identical to the precast decked bulb-tee sections, but with thinner 49-inch wide top flanges and no monolithic deck.

Girders with Cast-in-Place Decks
Design bridges with cast-in-place decks to be continuous for live load and superimposed dead loads by using a cast-in-place closure diaphragm at piers whenever possible. The design of the girders for continuous structures is similar to the design for simple spans except that, in the area of negative moments, treat the member as an ordinary reinforced concrete section. Assume that the members are fully continuous with a constant moment of inertia when determining both the positive and negative moments due to loads applied after continuity is established.

Loading Conditions
Consider the following five loading conditions in the design of a precast, prestressed girder:

1. The first loading condition is when tensioned strands are in the bed prior to placement of the concrete. Seating losses, relaxation of the strand, and temperature changes affect the stress in the strand prior to placement of the concrete. The fabricator must consider these factors during the fabrication of the girder and must adjust the initial strand tension to ensure that the tension prior to release meets the design requirements for the project. The prestressing shop drawings should present a discussion on the fabricator's proposed methods to compensate for seating losses, relaxation, and temperature changes.

2. The second loading condition is when the strands are released and the force is transferred to the concrete. After release, the girder will camber up and be supported at the girder ends only. Therefore, the region near the end of the member is not subject to bending stresses due to the dead load of the girder and may develop

Table 14-4
Standard Decked Bulb-Tee Top Flange
Required Reinforcing Steel

	Bar	Girder Width					
		≤ 6.0′	6.0′ - 6.5′	6.5′ - 7.0′	7.0′ - 7.5′	7.5′ - 8.0′	8.0′ - 8.5′
Interior	G401 Spaced at	8″	7″	6.5″	6″	5.75″	5.5″
	G402 No. of Bars	8	10	10	10	12	14
	G501 Spaced at	8″	7″	6.5″	6″	5.75″	5.5″
	G502 No. of Bars	8	10	10	10	12	14
Exterior	G401X Spaced at	7″	6″	5.5″	5″	4.5″	4″
	G402X No. of bars	8	10	12	14	16	16
	G501X Spaced at	7″	6″	5.5″	5″	4.5″	4″
	G502X No. of bars	8	10	12	14	16	16

tensile stresses in the top of the girder large enough to crack the concrete. The critical sections for computing the critical temporary stresses in the top of the girder should be near the end and at all debonding points. As an option, if the bridge engineer chooses to consider the transfer length of the strands at the end of the girder and at the debonding points, then assume that the stress in the strands is zero at the end of the girder or debonding point and vary linearly to the full transfer of force to the concrete at the end of the strand transfer length.

There are several methods to relieve excessive tensile stresses near the ends of the girder:

a. harping some of the strands to reduce the strand eccentricity at the end of the girder

b. debonding, where the strands remain straight but wrapped in plastic over a predetermined distance to prevent the transfer of prestress to the concrete through bonding

c. adding additional strands in the top of the girder that are bonded at the ends but are debonded in the center portion of the girder. These strands are typically detensioned after the girder is erected.

Use the level of effective prestress immediately after release of the strands, which includes the effects of elastic shortening and the initial strand relaxation loss, to compute the concrete stresses at this stage.

3. The third loading condition may occur a few weeks to a few months after strand release when the girder is transported.

4. The fourth loading condition may occur several weeks to several months after strand release when the girder is erected and a composite deck may be cast. Camber growth and prestress losses are design factors at this stage. If a cast-in-place composite deck is placed, field adjustments to the haunch thickness are usually needed to provide the proper vertical grade on the top of deck and to keep the deck thickness uniform. The bridge engineer needs reliable estimates of deflection and camber to prevent excessive haunch thickness or to avoid significant encroachment of the top of girder into the bottom of the concrete deck. Stresses at this stage are usually not critical.

   See Section 8.7 of the *PCI Bridge Design Manual* for determining the girder camber at erection.

5. The fifth loading condition is after an extended period of time during which all prestress losses have occurred and loads are at their maximum. This is often referred to as the "maximum service load, minimum prestress" stage. The tensile stress in the bottom fibers of the girder at mid-span generally controls the design. DOT&PF policy is to limit tensile stress in the

bottom fibers of the girder to 0 ksi under the Service limit state loading.

Flexural Resistance

The design of prestressed concrete members in flexure normally begins with the determination of the required prestressing level to satisfy service conditions. Consider all load stages that may be critical during the life of the structure from the time prestressing is first applied. Follow this by a strength check of the entire member under the influence of factored loads. The strength check seldom requires additional strands or other design changes.

Intermediate Diaphragms

Reference: LRFD Article 5.12.4.

Provide cast-in-place (CIP) concrete intermediate diaphragms at the point of maximum moment for all new precast concrete girder bridges. Design the girders to support the dead load of these diaphragms. Provide openings for both planned and future utilities. At a minimum, provide an 8-inch diameter opening in each exterior girder bay for future utilities.

Figure 14-5 presents the preferred (i.e., standard) details for the CIP intermediate concrete diaphragms. Present these details on the "Typical Section" sheet or on the "Girder Details" sheet in the contract documents. Show the openings in the interior girder webs and the inserts in the exterior girder webs on the "Girder Details" sheet.

For continuous precast, prestressed girder spans, the closure diaphragms at the piers must be cast monolithically or integrally with the deck slab.

Shear Keys and Shear Connectors

Provide shear keys and shear connectors consistent with the details shown in Figure 14-6. Locate the shear connectors at 4-foot spacing along the interior top flanges.

Responsibilities

1. **Bridge Engineer.** The bridge engineer is responsible for ensuring that the proposed design will work. Select a cross section with a center of gravity (force and location), and provide a strand/tendon size and pattern to achieve the required allowable Service limit state stresses and factored flexural resistance.

   The contract documents will specify the exact value with respect to the compressive strength that the contractor must reach at release (f'_{ci}) and at 28-days (f'_c). See Section 14.4.2.

   The bridge engineer is also responsible for a preliminary investigation of shipping and handling issues where larger or long precast girders are used or where unusual site access conditions are encountered. Contact girder fabricators if shipping and handling issues appear to be unusual.

2. **Contractor.** In general, the Contractor is responsible for implementing the prestressed concrete design according to the bridge engineer's plans and specifications. The Contractor will provide shop drawings and all necessary calculations (See Chapter 25 for shop drawing checklists).

   In addition, for precast girders, the Contractor is responsible for investigating stresses in the components during proposed handling, transportation, and erection. The Contractor may propose changes to the cross sectional shape of the girder. In these cases, the Contractor must redesign the girder to meet all requirements of the project. A registered civil/structural engineer licensed in Alaska must submit design calculations and drawings for approval.

14.4.4. Cast-in-Place, Post-Tensioned Box Girders

Reference: LRFD Articles 5.12.5.

Cast-in-place, post-tensioned box girders may be used for longer span (160 feet to 500 feet) application, if they prove economical. For design requirements, see the appropriate articles of the *LRFD Specifications* and the California Department of Transportation Amendments to the AASHTO *LRFD Bridge Design Specifications*.

14.4.5. Documentation for Prestressed Concrete Girder Design Calculations

Figure 14-7 presents selected design calculations for the superstructure portion of a precast decked bulb-tee bridge. This example also provides suggested style and format for documenting design calculations.

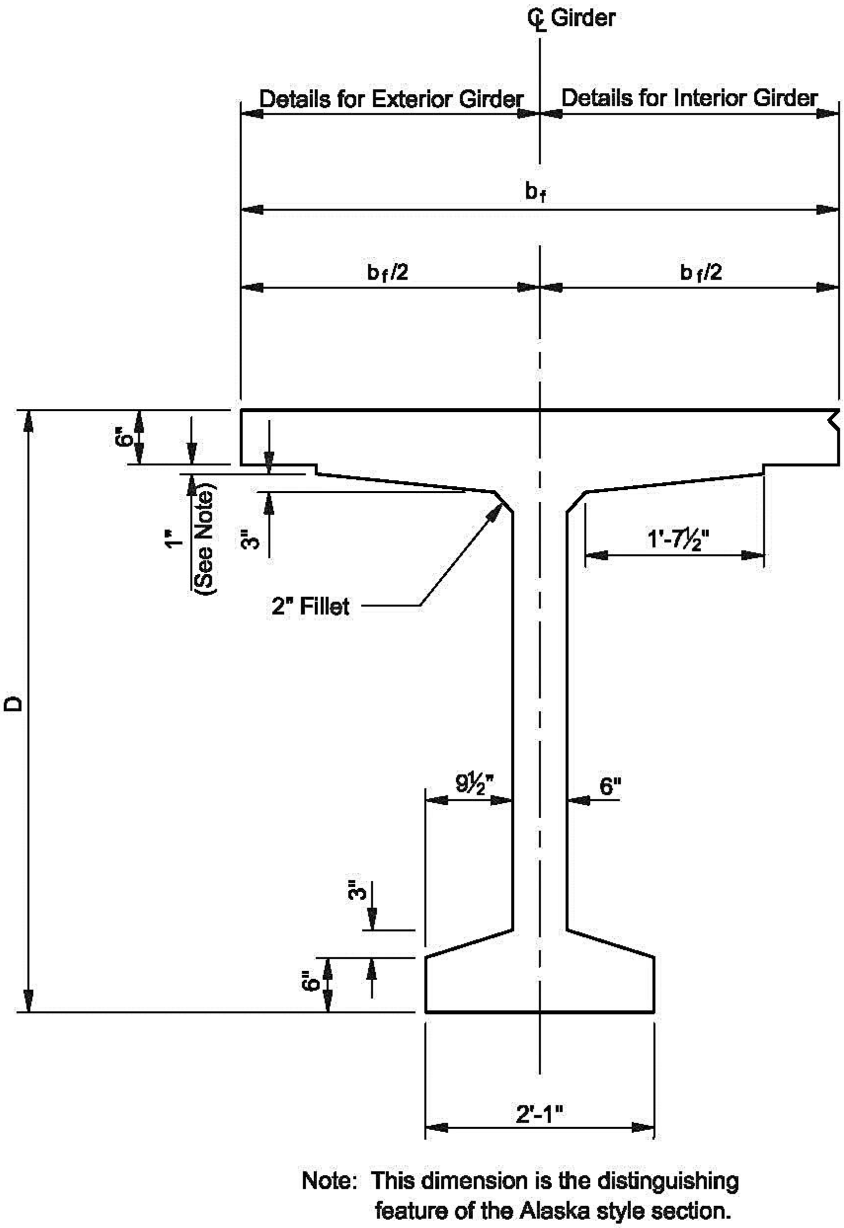


Figure 14-3
Standard Alaska-Style Precast Decked Bulb-Tee Section

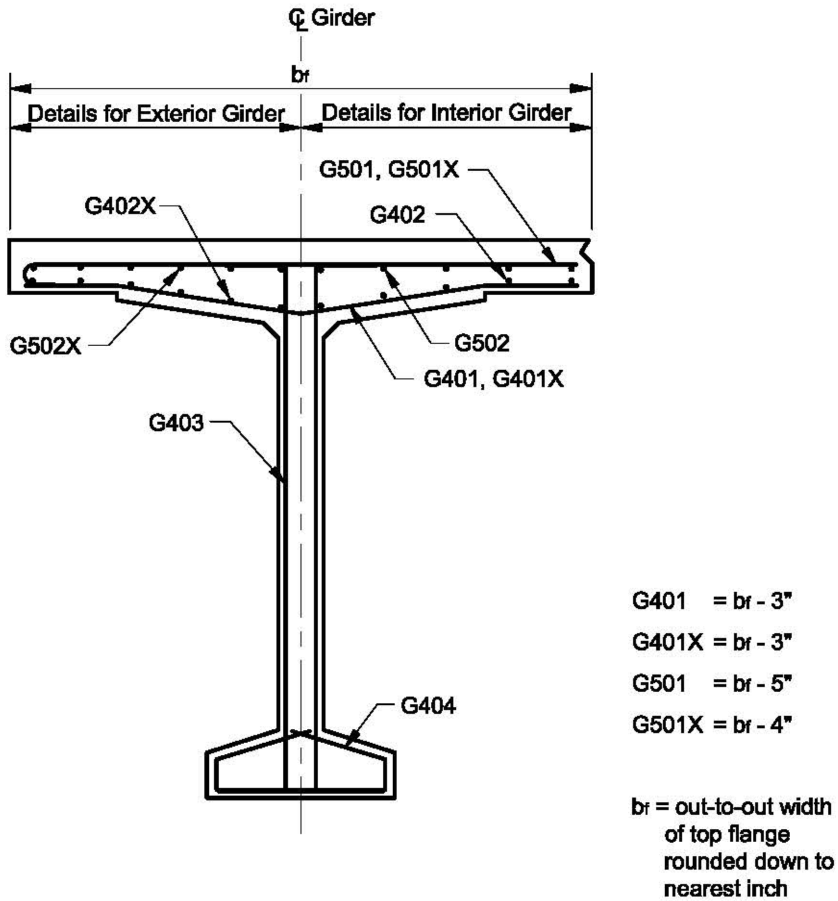
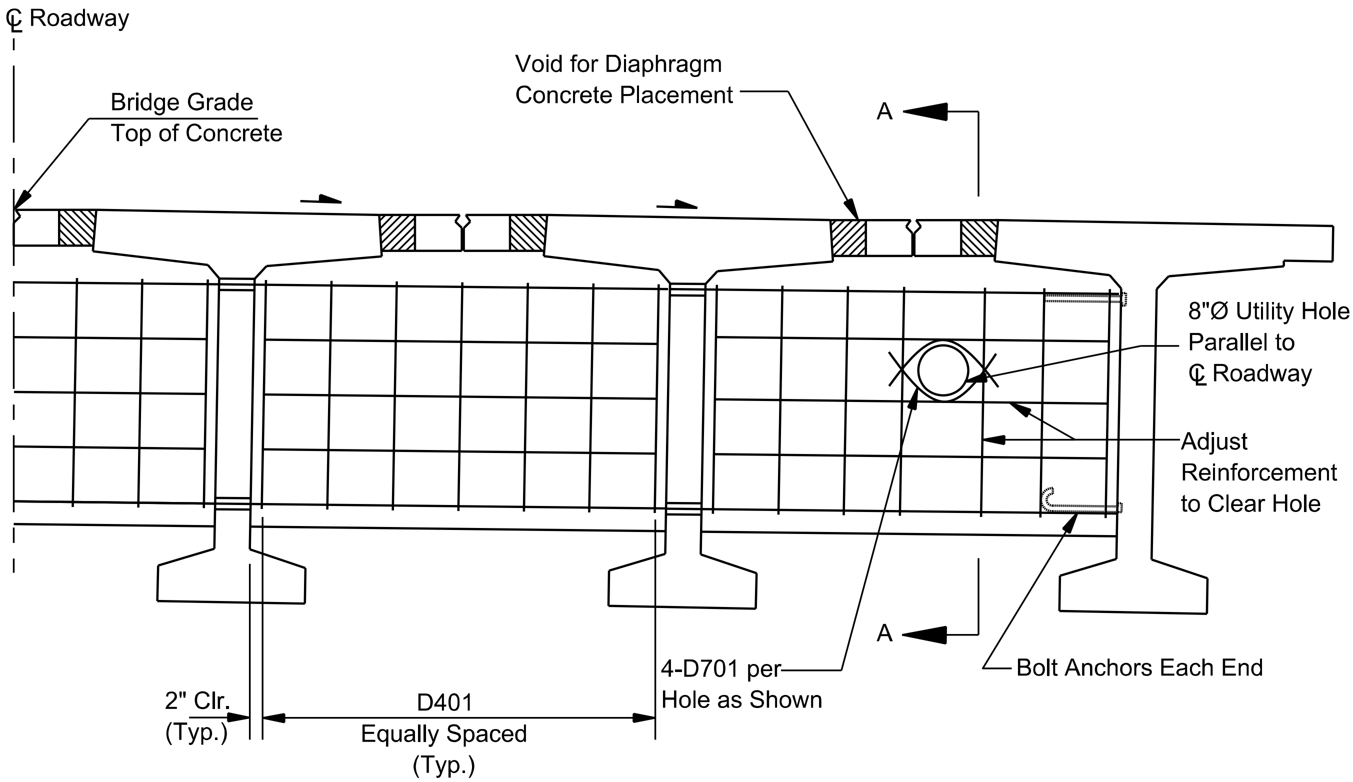
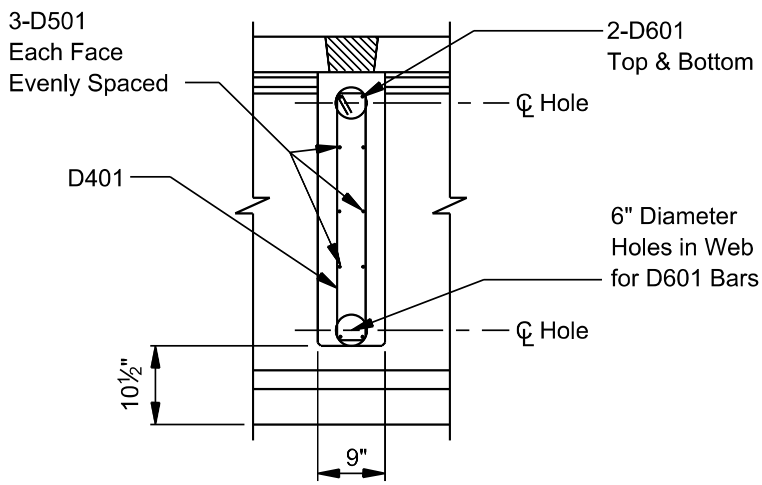


Figure 14-4
Precast Decked Bulb-Tee Typical Reinforcing Steel



Typical Section



Section A-A

Figure 14-5
Intermediate Concrete Diaphragm

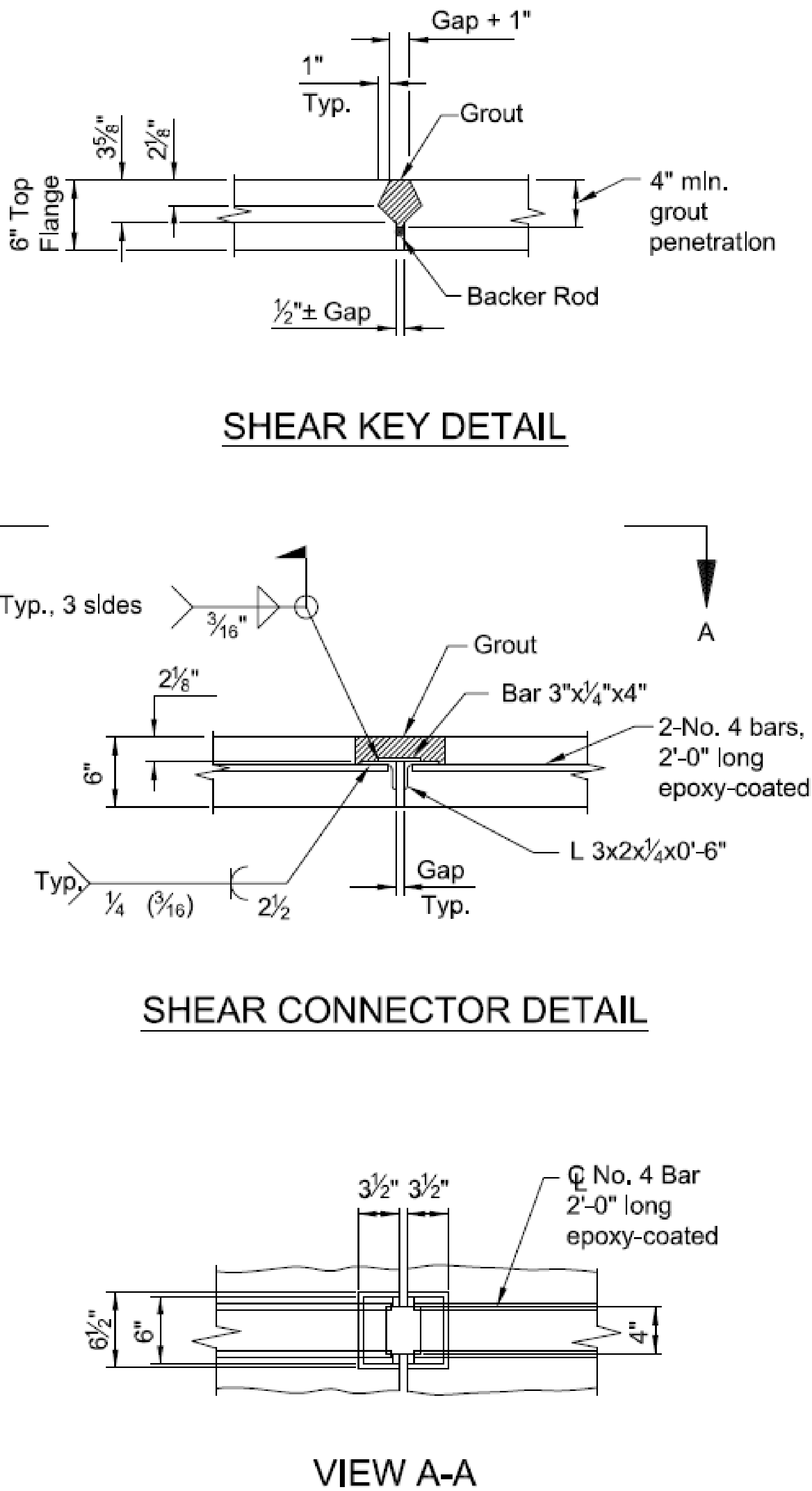


Figure 14-6
Shear Key and Shear Connector Details

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

For: ANTLER SLOUGH BRIDGE

LIVE LOADS

- For a simple span bridge with a 140.86 bearing-to-bearing length, the HL93 live load moment is:

$$M_{LL+IM} = M_{truck} \times IM + M_{LANE}$$

Where:

$M_{truck} = 2258$ k-ft

IM = Impact allowance

= 1.33

$$M_{LANE} = \frac{(0.64)(140.86)^2}{8} = 1587$$

So:

$M_{LL+IM} = (2258)(1.33) + 1587$ k-ft

$\therefore M_{LL+IM} = 4590$ k-ft ←

- The live load distribution factors calculated by the computer program are:

DF_V = 0.633

DF_M = 0.515

COMMENTS

AASHTO 3.6.1.2

AASHTO T3.6.2.1-1

Same as computer
✓ OK
Near midspan

Sheet 29 of ## Sheets

Figure 14-7
Sample Calculations
(Page 1 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

For: ANTLER SLOUGH BRIDGE

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

COMMENTS

- verify that the moment distribution factor is reasonable with the following approximation:

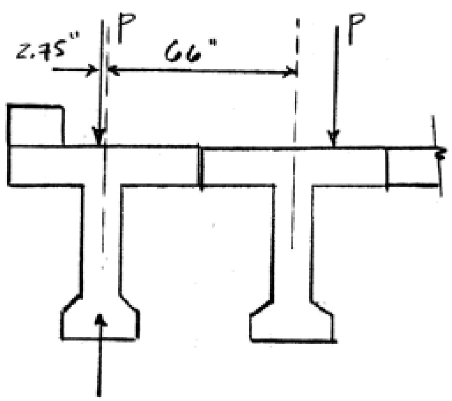
$$DF_M \cong \frac{1}{2} \times \frac{S}{5.5} \cong \frac{5.50 \text{ ft}}{11}$$

$\therefore DF_M \cong 0.500$ vs. 0.515 ✓ CE

Approximation based on "old" S-over method

3%

- and for the shear DF in the exterior girder,



AASHTO 4.6.2.2.1

$$R = \frac{68.75}{66} = 1.042\,P$$

$$DF_V = \frac{R \times m}{2P} = \frac{(1.042\,P)(1.2)}{(2)(P)}$$

$\therefore DF_V = 0.625$ vs. 0.633 ✓ CE

1%

Sheet 30 of ## Sheets

Figure 14-7
Sample Calculations
(Page 2 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

For: ANTLER SLOUGH BRIDGE

PRESTRESS LOSSES

COMMENTS

- Verify that the prestress losses calculated by the computer program are reasonable.

$$\Delta f_{PT} = \Delta f_{PES} + \Delta f_{PLT}$$

AASHTO 5.9.5.1

Where:

Δf_{PT} = Total short and long term loss

Δf_{PES} = Elastic shortening loss

$= \frac{E_P}{E_{c_i}} f_{cgp}$

AASHTO 5.9.5.2.3.a
See Sheet 24

E_P = 28500 ksi

E_{ci} = 5185 ksi

f_{cgp} = Concrete stress at C.G. of prestressing and DL self weight

$= \frac{-P_i}{A} - \frac{P_i \times e}{S_{cg}} + \frac{M_{DG1}}{S_{cg}}$

P_i = Initial P/S after Δf_{PES}

$= [(0.7)(270) - \Delta f_{PES}](0.153)(N_s)$

N_s = No. of strands = 64

See Sheet 25

A = 1013 in²

S_{cg} = 15577 in³

M_{PCI} = 2728 k-ft = 32736 k-in

e = 36.89 in

See Sheet 28

Sheet 31 of ## Sheets

Figure 14-7
Sample Calculations
(Page 3 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
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PUBLIC FACILITIES

Computations

For: ANTLER SLOUGH BRIDGE

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

Try, $\Delta f_{PES} = 19.1$ ksi

So, $P_i = (189 - 19.1)(0.153)(64) = 1663.7$

And,

$$f_{cgp} = \underbrace{\frac{-1663.7}{1013}}_{-1.642} - \underbrace{\frac{(1663.7)(36.89)}{15577}}_{-3.940} + \underbrace{\frac{32736}{15577}}_{+2.102}$$

$f_{cgp} = 3.480$ ksi [compression]

Then,

$$\Delta f_{PES} = \frac{28500}{5185}(3.480\text{ ksi})$$

✓OK

$\therefore \Delta f_{PES} = 19.1$ ksi -- Same as trial ←

COMMENTS

ALSO THE SAME AS THE COMPUTER $\therefore$ GOOD

Sheet 32 of ## Sheets

Figure 14-7
Sample Calculations
(Page 4 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

Computations

For: ANTLER SLOUGH BRIDGE

- and for the long-term P/S losses using the approximate method:

$$\Delta f_{PLT} = 33.0\left[1 - \frac{0.15\,(f'_c - 6)}{6}\right] + 6PPR - 8^{*}$$

Where:

$f'_c = 7.5$ ksi
$PPR = 1.0$ for P/S girders

So:

$$\Delta f_{PLT} = 33\left[1 - \frac{(0.15)(1.5)}{6}\right] - 2$$

$$\therefore \Delta f_{PLT} = 29.8 \text{ ksi} \leftarrow$$

- The total P/S losses are:

$$\Delta f_{PT} = 19.1 + 29.8$$

$$\therefore \Delta f_{PT} = 48.9 \text{ ksi} \leftarrow$$

COMMENTS

AASHTO T.5.9.5.3-1

* FOR LOW-RELAXATION STRAND

SAME AS COMPUTER VALUE

SAME AS PROGRAM

Sheet 33 of ## Sheets

Figure 14-7
Sample Calculations
(Page 5 of 6)

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

Computations

Project No. ______
Bridge No. 2165
Calc. by EEM Date 2/10/6
Checked by ______ Date ______

For: ANTLER SLOUGH BRIDGE

COMMENTS

- The prestress forces at various times:

Jacking

$f_j = (0.7)(270) = 189 \text{ ksi}$ ←

$P_j = (64)(0.153)(189) = 1851 \text{ k}$

Release

$f_i = 189 - 19.1 = 169.9 \text{ ksi}$ ←

$P_i = (64)(0.153)(169.9) = 1664 \text{ k}$

Final

$f_f = 169.9 - 29.8 = 140.1 \text{ ksi}$ ←

$P_f = (64)(0.153)(140.1) = 1372 \text{ k}$

Sheet 34 of ## Sheets

Figure 14-7
Sample Calculations
(Page 6 of 6)

14.5. References

Goodnight, J.C., Kowalsky, M.J., and Nau, J.M., 2015. *The Effects of Load History and Design Variables on Performance Limit States of Circular Bridge Columns*, volumes 1 and 2, Report No. 4000(72), Alaska DOT&PF, Juneau, AK, 278 and 748 pp.

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15. Structural Steel Superstructures

This chapter discusses structural steel provisions of the *LFRD Bridge Design Specifications* that require amplification, clarification, and/or an enhanced application. Section 11.5 of this *Manual* provides criteria for structural steel superstructure site selection. These criteria include span lengths, girder spacing, geometrics, aesthetics, and cost. Chapter 15 addresses specific DOT&PF practices for the design and detailing of steel superstructures.

DOT&PF typically uses structural steel superstructures where prestressed concrete, decked bulb-tee girder superstructures will not work for geometric, environmental, cost, or constructability reasons. As such, steel girders are selected where span lengths exceed 145 feet, where falsework needs to be minimized, and in remote locations where bulb-tee girders prove too heavy to transport and erect. Steel box girders have increased stability during erection in comparison to steel I-girders and may be appropriate for some locations. Rolled beam sections may prove more cost-effective than welded steel plate girders for shorter spans in remote locations.

15.1. General

15.1.1. Economical Steel Superstructure Design

Factors that influence the initial cost of a steel bridge include, but are not limited to, detailing practices, the number of girders, the grade of steel, type and number of substructure units (i.e., span lengths and arrangements), steel weight, fabrication, transportation, and erection. The cost associated with these factors changes periodically in addition to the cost relationship among the factors. Therefore, evaluate and modify these guidelines as necessary for each bridge to determine the most economical type of steel girder.

Based upon market factors, the availability of steel, particularly large rolled wide flange sections, may be an issue in meeting the construction schedule. The bridge engineer must verify the availability of the specified steel. Bridge designers should contact producers to ensure the availability of plates and rolled beams. For more detailed information on material (plates, rolled beams) availability, see Section 1.4 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructability*, G12.1.

Number of Girders/Spacing

See Section 11.4.5 for general information on the number of girders in girder bridges. See Section 11.5.3. for DOT&PF criteria on typical girder spacing for both I-girder and tub girder steel bridges. For detailed commentary on steel girder spacing, see Section 1.2 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructability*, G12.1. Typically, fewer deeper girders result in less steel weight; however, this approach may not result in the lowest superstructure cost.

Exterior Girders

The location of the exterior girder with respect to the overhang is controlled by these factors:

- Locate the exterior girder to limit the dead load and live load on girders such that the exterior girder does not control the design (i.e., the interior and exterior girder sections are identical). Overhang lengths equal to approximately 25 percent to 45 percent of the interior girder spacing typically yield moment balance between exterior and interior girders.
- The space required for deck drains may have an effect on the location of the exterior girder lines.
- Consider aesthetics when determining the location of the exterior girder lines.

Span Arrangements for Continuous Girders

Use span arrangements that balance the moments in the spans (i.e., equal maximum moments in end and interior spans). The end-span lengths are typically between 70 percent and 80 percent of the length of interior spans. As a result, the optimum proportions of the girder in all spans will be nearly the same, resulting in a more efficient and cost-effective design. To prevent uplift at the abutments, avoid end spans less than 70 percent of the interior spans.

15.1.2. Rolled Beams vs Welded Plate Girders

Welded Plate Girders

Design welded steel plate girders to optimize total cost including material costs while also considering fabrication and erection costs. Top flanges of composite plate girders are typically thinner than their bottom flanges. Vary the flange section along the length of the bridge, generally following the moment envelope to save cost by offsetting the increased fabrication costs of welded flange transitions with larger savings in material costs. Typically, the design should vary only flange thicknesses, not widths, within a field section to reduce fabrication costs. The webs of plate girders are typically deeper and thinner than the webs of rolled beams.

Due to buckling considerations, address the stability of the compression flange (i.e., the top flange in positive-moment regions and the bottom flange in negative-moment regions) by providing lateral-brace locations based upon calculations instead of the 25-foot diaphragm spacings of the AASHTO *Standard Specifications for Highway Bridges*. The traditional 25-foot diaphragm spacing, however, provides a good starting point.

On straight bridges (skewed or non-skewed), detail diaphragms as secondary members. On horizontally curved bridges, design diaphragms as primary members, because horizontally curved girders transfer a significant amount of load between girders through the diaphragms.

Rolled Beams

Rolled steel beams are doubly symmetrical sections with equal-dimensioned top and bottom flanges and relatively thick webs. Thus, these beams do not optimize the cross sections for weight savings (like a plate girder) but are cost effective due to lower fabrication and erection costs. The relatively thick webs preclude the need for intermediate web stiffeners, but full-height bearing stiffeners are required at all support locations. Unless difficult geometrics or limited vertical clearances control, rolled steel beam superstructures are more cost effective for relatively shorter spans (up to about 80 feet).

Rolled steel beams are more readily available in depths up to 36 inches, with deeper beams typically rolled less frequently. Before beginning final design, verify with one or more potential fabricators that the section size and length are available.

If a large size (greater than 36-inch deep) rolled-beam design is proposed for a new bridge, the contract documents should allow the substitution of a welded plate-girder design of equal flange and web plate sizes at the Contractor's discretion.

15.1.3. Economical Girder Proportioning

General

Make girders composite with the bridge deck through shear studs and continuous over interior supports where possible. For economy, all girders in a multi-girder bridge should be identical where possible or, if necessary, use a minimal number of plate sizes.

Haunched (Variable-Depth) Girders

When practical, use constant-depth girders (i.e., girders with constant web depths). Haunched girders are generally uneconomical for interior span lengths less than 240 feet. Consider using parabolic haunched girders where aesthetics or other special circumstances are encountered, but constant-depth girders will generally be more cost effective.

Flange Plate Sizes

The minimum flange plate size for welded girders is 12 inches by 1 inch to avoid cupping of the flanges due to distortion from welding. Use as wide a flange girder plate as practical, consistent with stress and b/t (flange width/thickness ratio) requirements. The wide flange contributes to girder stability during handling and in-service. As a guide, the flange width should be approximately 20 to 30 percent of the web depth. Provide flange widths in increments of 2 inches. The maximum flange thickness is 3 inches to ensure more uniform through-thickness properties. Avoid thicker plates; they demonstrate relatively poorer properties near mid-thickness.

Within a single field section (i.e., an individual shipping piece), provide flanges of constant width. A design of multiple identical girders with constant-width flanges minimizes fabrication costs.

Proportion flanges so that the fabricator can economically cut them from steel plate between 60 inches and 120 inches wide. The most economical mill widths are 72 inches, 84 inches, 96 inches, and 120 inches. Allow ¼ inch for internal torch cutting lines and ½ inch for exterior torch cutting lines; see Figure 15-1. Group flanges to provide an efficient use of the plates. Because structural steel plate is most economically purchased in these widths, it is advantageous to repeat plate thicknesses as much as practical. If practical, group plates of like width by thickness to meet the minimum width-purchasing requirement, but this economical purchasing strategy may not be possible for thicker, less-used plates.

The most efficient method to fabricate flanges is to groove-weld together several wide plates of varying thicknesses received from the mill. After welding and non-destructive testing, the individual flanges are "stripped" (i.e. cut longitudinally) from the full plate. This method of fabrication reduces the number of welds, individual runoff tabs for both start and stop welds, the amount of material waste, and non-destructive testing. The objective, therefore, is for flange widths to remain constant within an individual shipping length by varying material thickness as required. Figure 15-1 illustrates one example of an efficient fabrication for girders.

Constant flange width within a field section may not always be practical in girder spans over 300 feet where a flange width transition may be required in the negative bending regions. Though not preferred, if a transition in width must be provided, shift the butt splice a minimum of 3 inches from the transition into the narrower flange plate. See Figure 15-2. This 3-inch shift makes it simpler to fit run-off tabs, weld and test the splice, and then grind off the run-off tabs.

For additional information on sizing flange plates, see Section 1.5 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructability*, G12.1.

Field Splices

Use field splices to reduce shipping lengths, but minimize their number because they are expensive. The preferred maximum length of a field section is 80 feet; however, lengths up to 175 feet are possible, but do not use field sections greater than 120 feet without considering shipping, erection, and site constraints. For remote locations, a maximum length of 120 feet may feasible, but should be verified. As a rule, the unsupported length in compression of the shipping piece divided by the minimum width of the flange in compression in that piece should be less than approximately 85. Good design practice is to reduce the flange cross sectional area by no more than approximately 25 percent of the area of the heavier flange plate at field splices to reduce the build-up of stress at the transition. For continuous spans, all of the field sections that cantilever over a pier should be of constant length to simplify erection.

Although steel girder segments are typically lighter than concrete, the bridge designer should contact the Alaska Measurement Standards and Commercial Vehicle Enforcement (MS&CVE) to verify feasibility of transporting overweight, overheight, and overwidth bridge components.

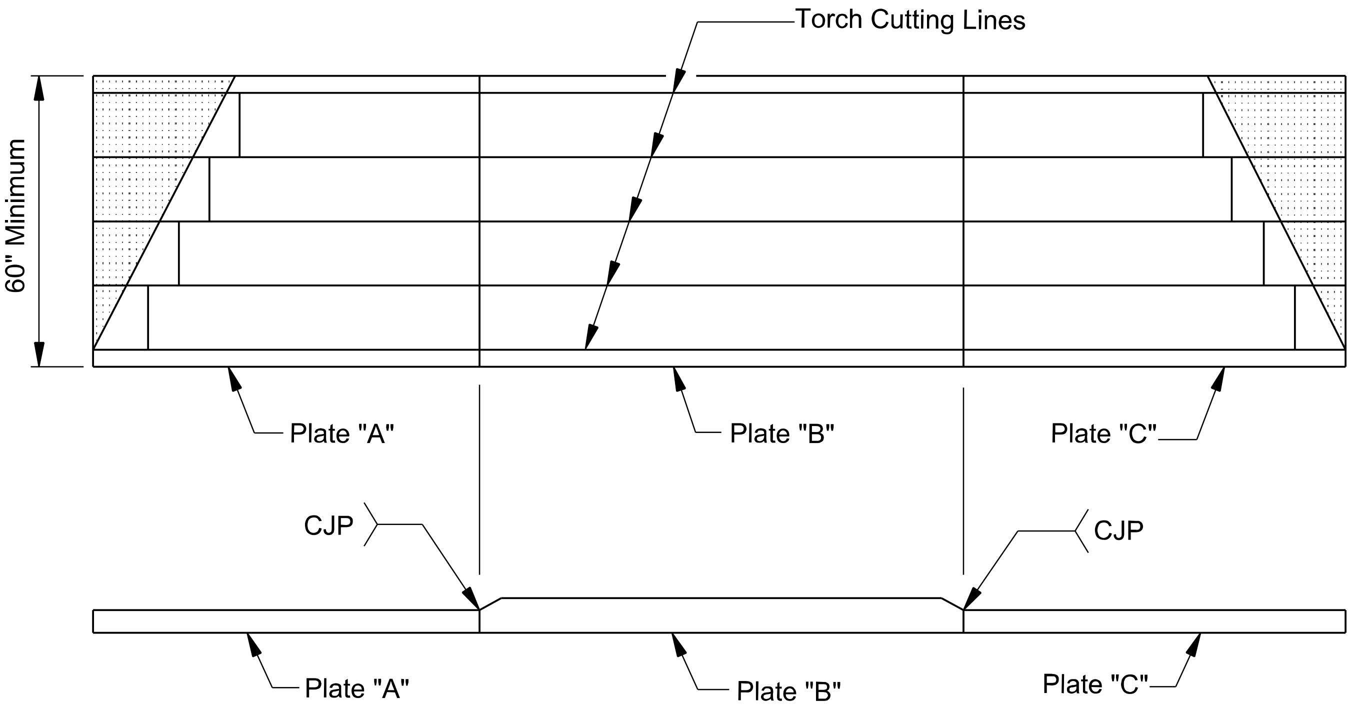

Figure 15-1
Grouping Flanges for Efficient Fabrication
(From the AASHTO/NSBA Steel Bridge Collaboration)

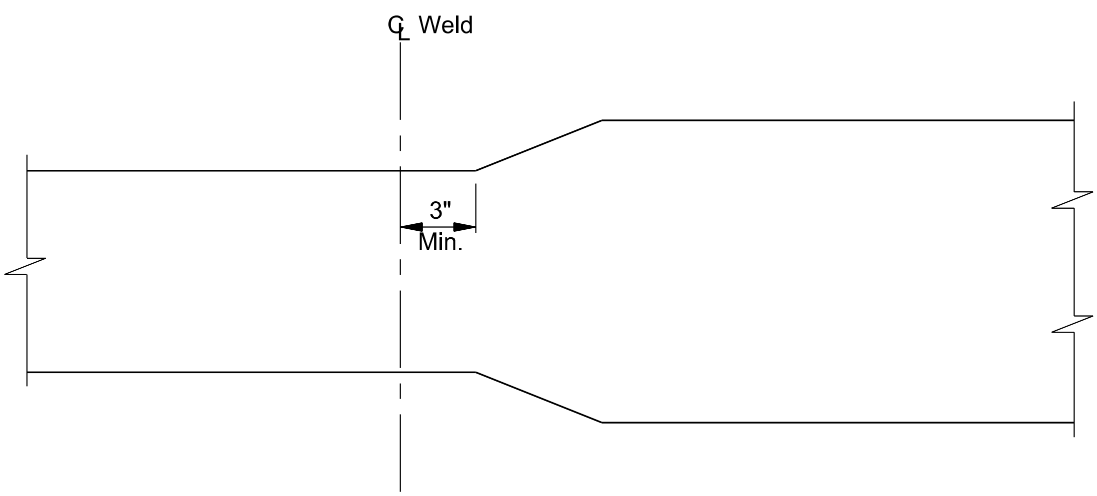

Figure 15-2
Flange Width Transition (Plan View)

Shop Splices
Include no more than two shop flange splices in the top or bottom flange within a single field section. In determining the points where changes in plate thickness occur within a field section, the designer should weigh the cost of groove-welded splices against extra plate area. Table 1.5.4-1 of the AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design for Constructability*, G12.1, provides guidelines for weight savings for Grade 50 steel required to justify a flange shop splice.

In many cases, it may be advantageous to continue the thicker plate beyond the theoretical step-down point to avoid the cost of the groove-welded splice.

To facilitate testing of the weld, locate flange shop splices at least 2 feet from web splices, and locate flange and web shop splices at least 6 inches from transverse stiffeners.

Section 1.5 of the AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design for Constructability*, G12.1, provides additional guidance on shop splices.

Web Plates
Where there are no depth restrictions, optimize the web depth and girder cross section. Do not change web thickness at any splice by less than 1/16 inch. Maintain symmetry by aligning the centerlines of the webs at splices.

Web design can have a significant impact on the overall cost of a plate girder. Considering material costs in isolation, make girder webs as thin as design considerations will permit. However, this practice will not always produce the greatest economy because fabricating and installing transverse stiffeners is one of the most labor-intensive of shop operations. The following guidelines apply to the use of transverse stiffeners:

- Unstiffened webs are generally more economical for web depths approximately 48 inches or less.

- For web depths between 48 inches and 72 inches, consider options for a partially stiffened or an unstiffened web, with unstiffened webs preferred. A partially stiffened web is one where the web thickness is proportioned to be 1/16 inch less than that allowed by specification for an unstiffened web at a given depth and, therefore, stiffeners are required only in areas of higher shear.

- Above 72 inches, consider options for partially stiffened or fully stiffened webs, with partially stiffened webs preferred. A fully stiffened web is one where stiffeners are present throughout the span.

Transverse Stiffeners
Flat bars (i.e., bar stock rolled to widths up to 8 inches at the mill) are typically more economical than plates for stiffeners. The stiffeners can be fabricated by shearing flat bars of the specified width to length. Proportion stiffeners that are intended to be fabricated from bars in ¼-inch increments in width and in ⅛-inch increments in thickness. Consult a fabricator for available flat bar sizes.

Longitudinally Stiffened Webs
Do not use longitudinally stiffened webs without the Chief Bridge Engineer's approval.

Minimum Thicknesses
The minimum thicknesses for components of structural steel plate girders are:

- webs: ½ inch
- flanges: 1 inch
- bearing stiffener plates: 1 inch
- gusset plates: ⅜ inch
- angles/channels: ¼ inch

15.1.4. Falsework
Design steel superstructures without intermediate falsework during the placing of the concrete deck slab. Shored construction is not permitted.

15.1.5. AISC Certification Program
The AISC Certification Program for Structural Steel Bridge Fabricators has three levels of expertise: Standard for Steel Bridges (Simple), Intermediate Bridge, and Advanced Bridge. For all bridge members, except rolled shapes, DOT&PF requires Advanced Bridge level certification with a Fracture Critical Endorsement.

Currently, DOT&PF does not require certification for sign structures.

15.1.6. Buy America
The "Buy America" provisions for Federal-aid projects are presented in 23 CFR Part 635.410. These provisions require that manufacturing processes for

steel and iron products and their coatings must occur in the United States. It is acceptable to use a minimal amount of foreign material if it does not exceed 0.1 percent of the total contract price or $2,500, whichever is more.

15.2. Materials

Reference: LRFD Article 6.4

15.2.1. Structural Steel

Reference: LRFD Article 6.4.1

The following presents typical DOT&PF practices for the material type selection for structural steel members. Note that, for ASTM A709 and AASHTO M270, the grades are common to both designations.

Grade 36

Grade 36 steel is typically only used for secondary structural members on straight girder bridges, such as:

- transverse stiffeners,
- diaphragms,
- sole plates, and
- bearing plates.

Grade 36 steel is becoming less used and, thus, less available at times. Generally, there is little or no cost difference between Grade 50 and Grade 36 steel. Wherever possible, allow substitution of Grade 50 for Grade 36 steel.

Grade 50 and 50W

Grade 50 steel is the most commonly used grade. It is typically used for the primary and secondary members such as:

- rolled beams,
- welded girders,
- splice plates,
- diaphragms,
- bearing plates,
- sole plates, and
- stiffeners.

High-Performance Steel

Do not use Grade HPS70W and HPS100W steel without the Chief Bridge Engineer's approval.

Corrosion-Protection Coatings

The following summarizes DOT&PF policies for the use of corrosion-protection coatings:

1. **Remote Sites**. Use metalizing (spray thermal metal coating) or hot-dip galvanizing for all steel bridges to be erected in remote sites to provide a relatively maintenance-free coating. Use metalizing where field sections are greater than 50 feet in length. For field sections less than 50 feet in length, hot-dip galvanizing may be used. See Chapter 20 for more discussion.
2. **All Other Sites**. For all other steel bridges, use galvanized steel with or without a painted top coat unless site conditions suggest that unpainted weathering steel will work. See FHWA Technical Advisory T5140.22 "Uncoated Weathering Steel in Structures," October 3, 1989. Historically, DOT&PF has only considered the use of unpainted weathering steel (i.e., Grade 50W) in the interior Alaska environment, which is dry.

The use of unpainted weathering steel is not appropriate in all environments and at all locations. The most prominent disadvantage of weathering steel is that it does not work well in coastal maritime environments. The inevitable staining of the substructure prior to the placement of the bridge deck due to weathering of the unpainted steel creates a poor image (i.e., one of lack of proper maintenance) to the traveling public. In addition, do not use weathering steel at locations where the following conditions exist:

1. **Environment**. Do not use in industrial areas where concentrated chemical fumes may drift onto the structure, or where the nature of the environment is otherwise undesirable.
2. **Water Crossings**. Do not use over bodies of water where the clearance over the ordinary high water is 10 feet or less.
3. **Grade Separations**. Do not use for highway grade separation structures.
4. **Coastal**. Do not use over saltwater or within 100 miles of the coast.

For additional guidance on the appropriate application and detailing of unpainted weathering steel, see the AISI publication "Performance of Weathering Steel in Highway Bridges: A Third Phase Report."

Charpy V-Notch Fracture Toughness

Reference: LRFD Article 6.6.2

Apply Temperature Zone 3 when using LRFD Table 6.6.2.1-2 for the State of Alaska.

All steel members require charpy V-notch fracture-toughness verification, and must be tested for non-fracture-critical (T) impact testing requirements unless fracture-critical (F) impact testing requirements are specified.

15.2.2. Bolts

Reference: LRFD Article 6.4.3

Type

For normal construction, high-strength bolts will be:

- **Galvanized or Metalized Steel**: Use ⅞-inch or 1-inch ASTM F3125, Grade A325, Type 1 mechanical galvanized with galvanized direct tension indicating washers (DTI).
- **Weathering Steel**: Use ⅞-inch or 1-inch ASTM F3125, Grade A325, Type 3 with weathering DTIs.

Twist-off bolts may be used, but DTIs are still required.

Hole Size

Typically, use a standard hole size. Do not use oversized or slotted holes, except in unusual circumstances. For galvanized girders, use standard holes and ream to the original size after galvanizing.

15.2.3. Splice Plates

In all cases, for all splice and filler plates, use the same steel as used in the web and flanges of plate girders.

15.3. Horizontally Curved Members

Reference: LRFD Articles 6.10 and 6.11

15.3.1. General

Wherever practical use chorded straight girders for curved alignments. Do not use curved girders, unless approved by the Chief Bridge Engineer.

The *LRFD Specifications* includes horizontally curved girders as a part of the provisions for proportioning I-shaped and tub girders at both the Strength and Service limit states. In addition, the *LRFD Specifications* specify analysis methodologies that detail various required levels of analysis.

15.3.2. Diaphragms, Bearings and Field Splices

The use of horizontally curved steel members requires the consideration of many factors that differ from straight girders including but not limited to:

1. **Cross Frames/Diaphragms**. All curved steel simple-span and continuous-span bridges must have diaphragms directed radially except end diaphragms, which should be placed parallel to the centerline of bearings. Cross frames and diaphragms should be as close as practical to the full depth of the girders.

2. **Load Considerations**. Design all diaphragms, including their connections to the girders, to carry the total transferred load at each diaphragm location. Design cross frame and diaphragm connections for the 75 percent and average load provisions of LRFD Article 6.13.1, unless actual forces in the connections are determined from an appropriate structural model. Using the provisions of LRFD Article 6.13.1 may result in very large connections that are difficult to detail.

3. **Charpy V-Notch Testing**. Consider cross frames and diaphragms to be primary members.

4. **Expansion/Contraction**. Bridges expand and contract in all directions. For typical bridges that are long in relationship to their width (say 2½ times the width), ignore the transverse expansion. For ordinary geometric configurations where the bridge length is long relative to the bridge width and the degree of curvature is moderate (those satisfying the requirements of LRFD Article 4.6.1.2.4b), no additional consideration is necessary for the unique expansion characteristics of horizontally curved structures. Wide, sharply curved, or long-span structures may require the use of high-load, multi-rotational bearings. Consider providing restraint either radially and/or tangentially to accommodate the transfer of seismic forces and the thermal movement of the structure because the bridge tries to expand in all directions.

5. **Flange Splices**. Design the splices in flanges of curved girders to carry lateral bending stresses and vertical bending stresses in the flanges.

15.4. Fatigue Considerations

Reference: LRFD Article 6.6

LRFD Article 6.6.1 categorizes fatigue as either "load induced" or "distortion induced." Load induced is a "direct" cause of loading. Distortion induced is an "indirect" cause in which the force effect, normally transmitted by a secondary member, may tend to change the shape of or distort the cross section of a primary member.

15.4.1. Load-Induced Fatigue

General

LRFD Article 6.6.1.2 provides the framework to evaluate load-induced fatigue, which is determined by:

- the stress range induced by the specified fatigue loading at the detail under consideration;
- the number of repetitions of fatigue loading a steel component will experience during its 75-year design life. For lower truck-traffic volume bridges, such as those that occur in Alaska, this is determined by using the actual anticipated truck volumes; and
- the nominal fatigue resistance for the Detail Category being investigated.

DOT&PF Policy

Where practical, design for infinite life. Avoid the use of steel bridge details with fatigue resistances lower than Detail Category C′ (i.e., Detail Categories D, E, and E′). Do not provide empty bolt holes.

The Chief Bridge Engineer must approve any exceptions to DOT&PF policy on load-induced fatigue.

15.4.2. Distortion-Induced Fatigue

LRFD Article 6.6.1.3 provides specific detailing practices for transverse and lateral connection plates intended to reduce significant secondary stresses that could induce fatigue crack growth. The provisions of the *LRFD Specifications* are prescriptive and require no mathematical computation of stress range.

15.4.3. Other Fatigue Considerations

In addition to the considerations in Sections 15.4.1 and 15.4.2, investigate the fatigue provisions in other Articles of Chapter 6 of the *LRFD Specifications*. These include:

- Fatigue and Fracture Limit State — LRFD Articles 6.5.3, 6.10.5, and 6.11.5.
- Fatigue at shear connectors — LRFD Articles 6.10.10.1.2 and 6.10.10.2.
- Bolts subject to axial-tensile fatigue — LRFD Article 6.13.2.10.3.

15.5. General Detail Requirements

Reference: LRFD Article 6.7

15.5.1. Deck Haunches

See Section 16.2.3 for design and detailing of haunches.

15.5.2. Camber

Where dead load deflection and vertical curve offset are greater than the minimum of 3/4 inch or 1/8 inch per 10 feet of segment length, provide girders with a compensating camber. Camber the entire girder length as required by the loading and profile grade. Calculate camber to the nearest 0.01 inch, with ordinates at variable locations as needed throughout the length of the girder. Show the required camber values from a chord line that extends from point of support to point of support.

Provide a camber diagram in all contract documents with structural steel girders. Include a camber diagram with ordinates as follows:

- steel DL deflection (DC),
- non-composite externally applied DL deflection (DC),
- superimposed DL deflection (DC),
- wearing surface deflection (DW) and
- total DL deflection,
- geometric camber.

The designer may show any additional information if desired (e.g., ordinates at field splices).

15.5.3. Diaphragms and Cross Frames

Reference: LRFD Articles 6.7.4 and 6.6.1.3.1

Diaphragms and cross frames are vitally important in steel girder superstructures. They stabilize the girders in the positive-moment regions during construction and in the negative-moment regions after construction. Diaphragms and cross frames also serve to distribute gravitational, centrifugal, and wind loads. Determine the spacing of diaphragms and cross frames based upon the provisions of LRFD Article 6.7.4.1. As with most aspects of steel girder design, the design of the spacing of diaphragms and cross frames is iterative. A good starting point is the traditional diaphragm and cross frame spacing of 25 feet with cross frames at the maximum and minimum movement locations. Most economical steel girder designs will use spacings typically greater than 25 feet in the positive-moment regions.

The following applies to diaphragms and cross frames:

1. **Location**. Place diaphragms or cross frames at each support and throughout the span at an appropriate spacing. Plan the location of the field splices to avoid conflict between the connection plates of the diaphragms or cross frames and any part of the splice material.
2. **Skew**. For up to a skew angle of 20 degrees, all intermediate diaphragms and cross frames may be placed perpendicular to the girders. Locating diaphragms and cross frames near girder supports on bridges with high skews requires careful consideration. When locating a diaphragm and cross frame between two girders, the relative stiffness of the two girders must be similar. Otherwise, the diaphragm and cross frame will act as a primary member supporting the more flexible girder. This may be unavoidable on bridges with exceptionally high skews where a rational analysis of the structural system will be required to determine actual forces.
3. **End Diaphragms**. Place end diaphragms along the centerline of bearing. Set the top of the diaphragm below the top of the girder to accommodate the joint detail and the thickened slab at the end of the superstructure deck, where applicable. Design the end diaphragms to support the edge of the slab including live load plus impact.
4. **Interior Support Diaphragms and Cross Frames**. Generally, place interior support diaphragms and cross frames along the centerline of bearing. They provide lateral stability for the bottom flange and bearings.
5. **Curved-Girder Structures**. Design diaphragms or cross frames connecting horizontally curved girders as primary members and orient them radially.
6. **Detailing**. Typically, detail diaphragms and cross frames to follow the cross slope of the deck; i.e., the diaphragm or cross frame is parallel to the bottom of the deck. This allows the fabricator to use a constant drop on each connection plate (i.e., the distance from the bottom of the flange to the first bolt hole on the connection plate is constant). The contract

documents should allow the contractor to use diaphragms or cross frames fabricated as a rectangle (as opposed to a skewed parallelogram). In this case, the drops vary across the bridge.

The following identifies typical DOT&PF practices on the selection of diaphragms and cross frames:

1. **Intermediate Diaphragms**. Preferably, use X-frames as intermediate diaphragms in I-girder bridges as shown in Figure 15-3. For relatively wide girder spacings relative to the girder depth (i.e., aspect ratio ≥ 1.5), a K-frame (as shown in Figure 15-4) may be more appropriate than an X-frame for intermediate diaphragms. Figure 15-5 shows a typical intermediate diaphragm for a box girder bridge, where a channel and the transverse stiffeners act as an intermediate diaphragm, which aids in fabrication and construction.

2. **End Diaphragms**. Preferably, use solid diaphragms as end diaphragms in I-girder bridges where possible, as shown in Figure 15-6. Figures 15-7 and 15-8 illustrate typical end diaphragms for box girder bridges for a section at an abutment and a section over a pier, respectively.

15.5.4. Lateral Bracing

Reference: LRFD Article 6.7.5

Lateral bracing typically consists of trussing members oriented in the horizontal plane placed along the bottom portion of steel I-girder bridges. The *LRFD Specifications* requires that the need for lateral bracing be investigated for all stages of assumed construction procedures. If the bracing is included in the structural model used to determine force effects, then design the lateral bracing for all applicable limit states.

In general, lateral bracing is not required in the vast majority of steel I-girder bridges (short through medium spans); however, the designer must check this. Typical diaphragms and cross frames will transfer lateral loads adequately to eliminate the need for lateral bracing.

For box girders, internal top lateral bracing is more typical. Employ steel stay-in-place forms attached with a structural weld as lateral bracing, with approval. See Figure 15-8.

LRFD Article 4.6.2.7 provides various alternatives relative to lateral wind distribution in multi-girder bridges.

15.5.5. Inspection Access (Box Girders)

Detail all new steel box girder bridges with access openings to allow inspection of the girder interior. Provide one access opening at each end of all box girders. Do not locate access openings over travel lanes or railroad tracks and, preferably, not over shoulders. Locate these such that the general public cannot gain easy entrance.

Access openings are typically placed in the webs of box girders; however, they may be placed in the bottom flange provided the total factored flange stress is less than 10 ksi. The dimensions of the access opening should be a minimum 30 inches by 30 inches square.

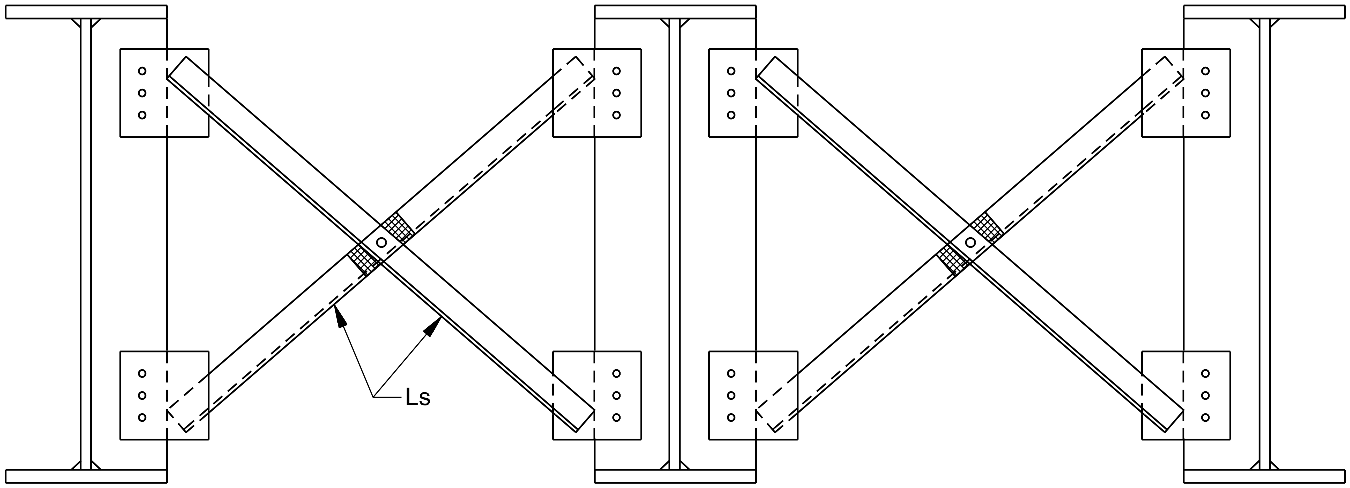

Figure 15-3
Typical Intermediate Diaphragm for an I-Girder Bridge (X-Frame)

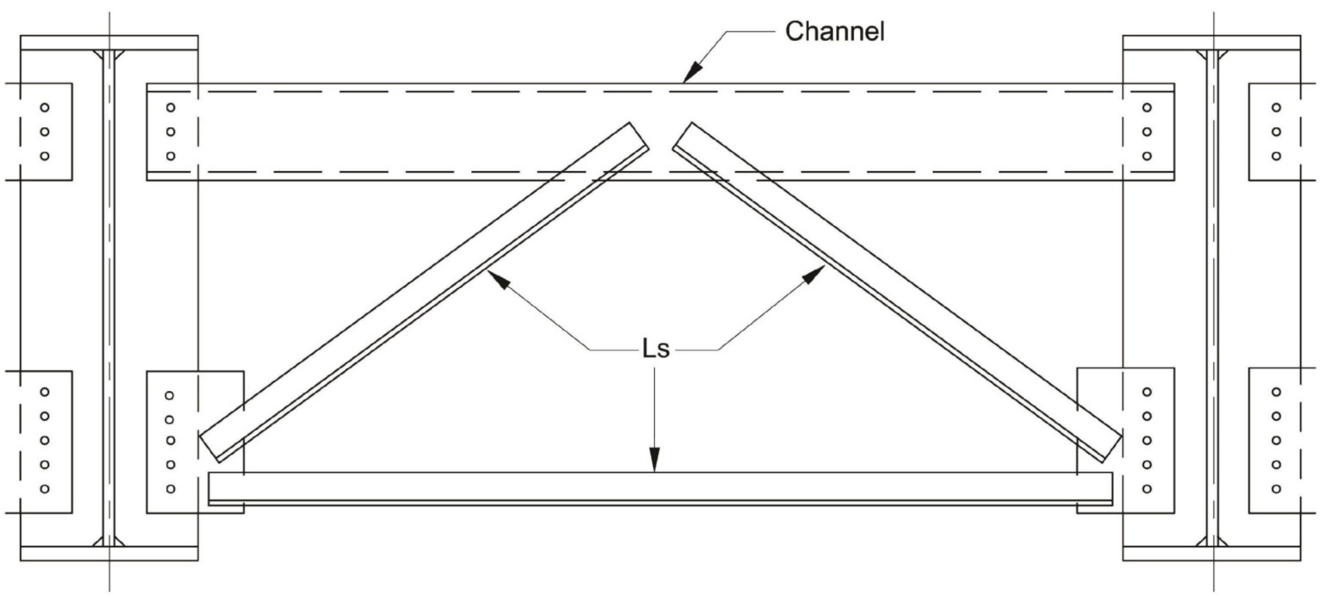

Figure 15-4
Typical Intermediate Diaphragm for an I-Girder Bridge (K-Frame)

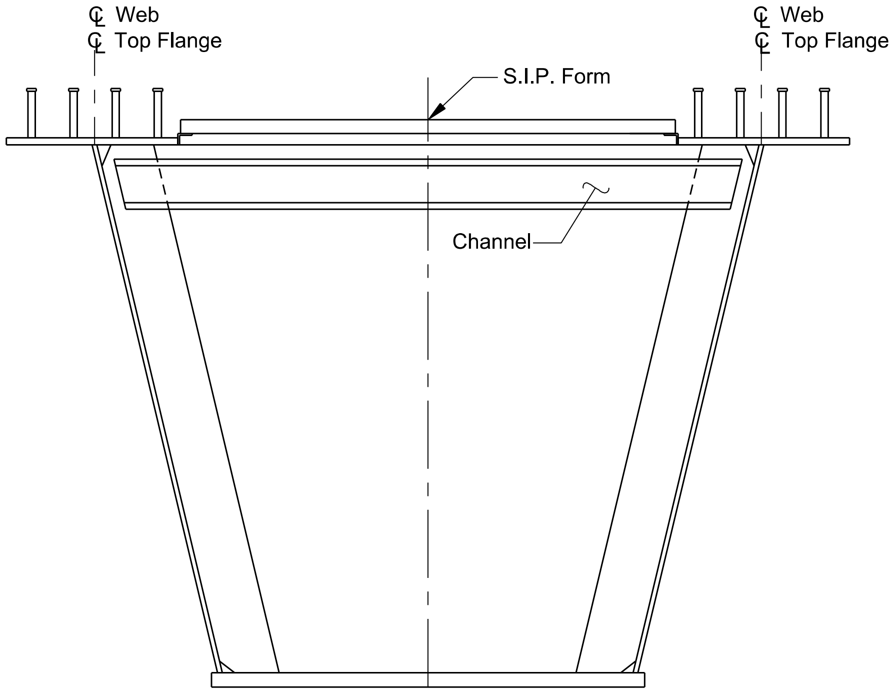

Figure 15-5
Typical Intermediate Diaphragm for a Straight, Square Box-Girder Bridge

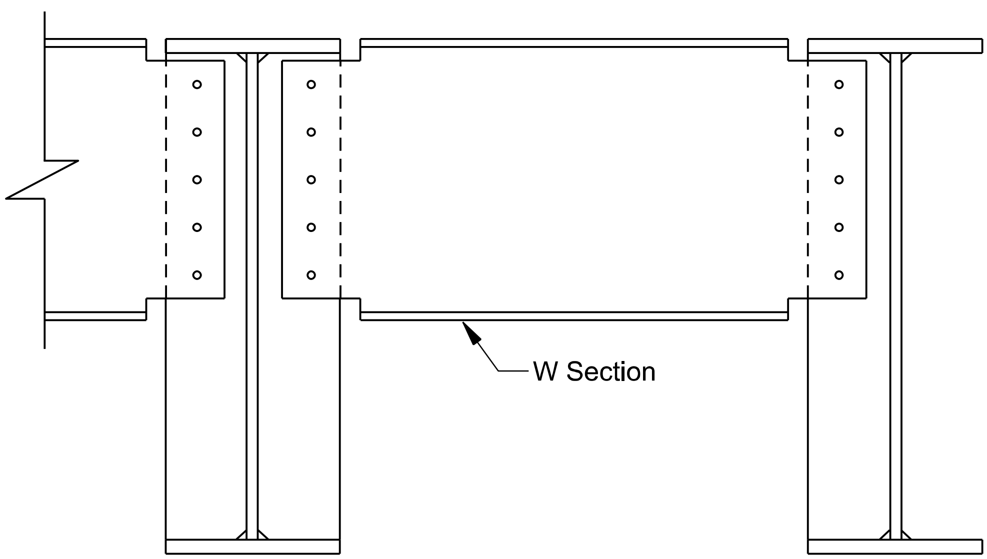

Figure 15-6
Typical End Diaphragm for a Rolled Beam or Shallow Plate I-Girder Bridge

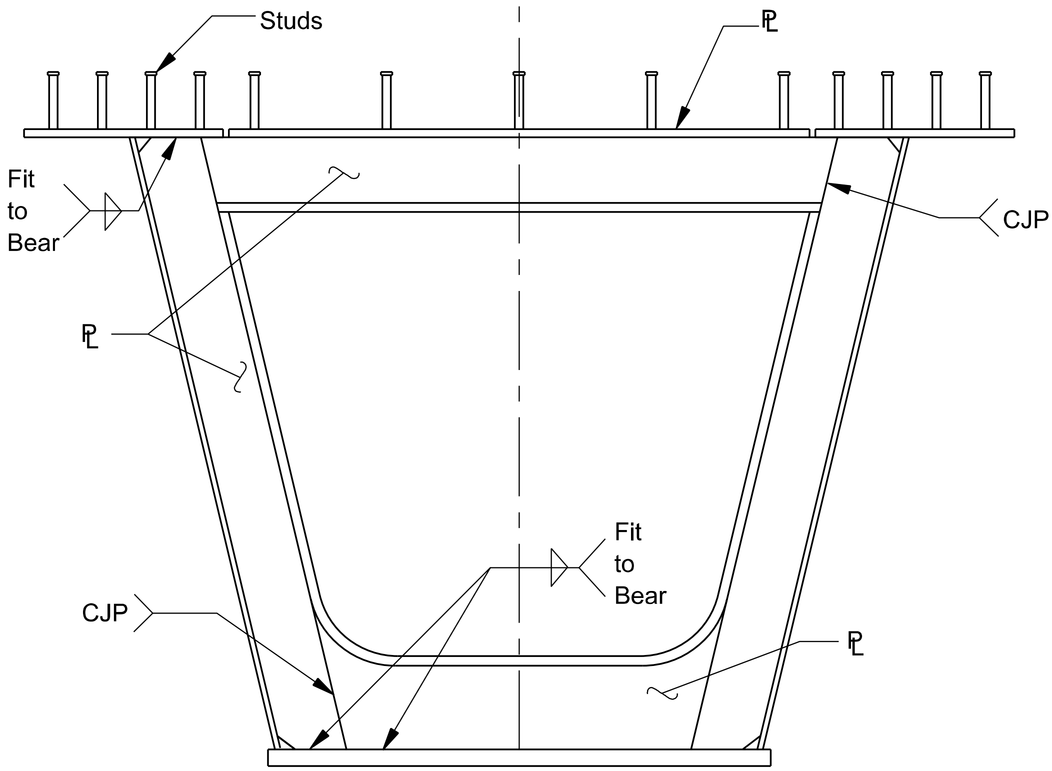

Figure 15-7
Typical End Diaphragm for a Box-Girder Bridge at an Abutment

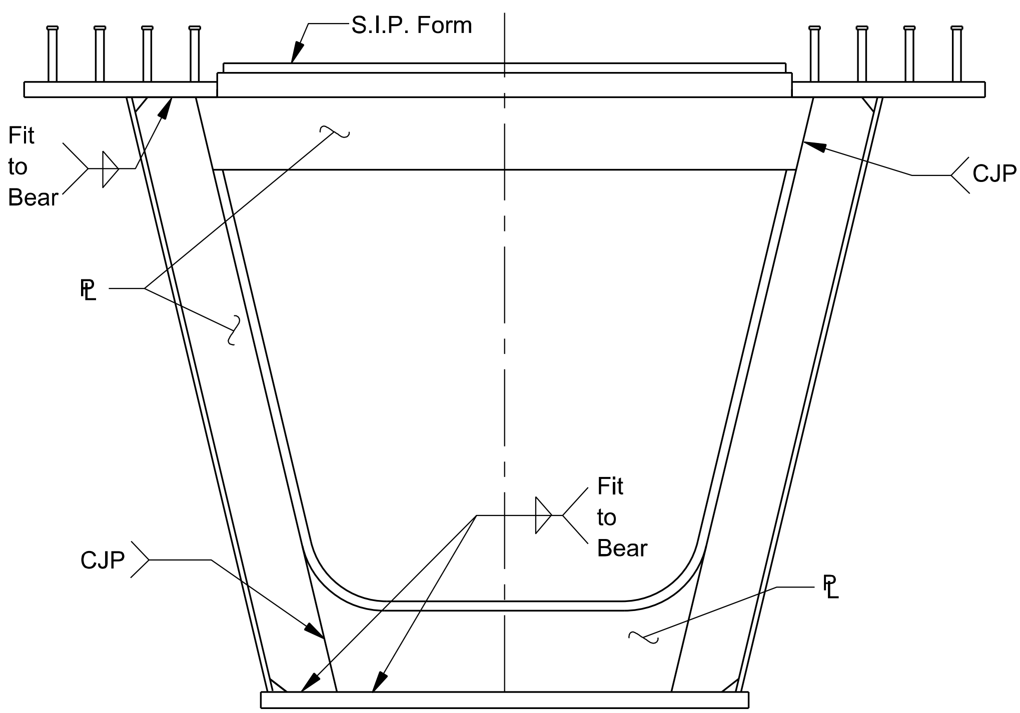

Figure 15-8
Typical End Diaphragm for a Box-Girder Bridge over a Pier

15.6. I-Sections in Flexure

Reference: LRFD Article 6.10

15.6.1. Limit States

Reference: LRFD Article 6.10.1

Positive-Moment Region Maximum-Moment Section

For a composite girder, consider the positive-moment region maximum-moment section to be compact in the final condition (see LRFD Article 6.10.7.1). The cured concrete deck in the positive-moment region provides a large compression flange that laterally braces the top flange. Very little, if any, of the web is in compression, but it should be checked.

Top Flange (Compression Flange). In the final condition after the deck has cured, the top flange adds little to the resistance of the cross section. During curing of the concrete deck, however, the top flange is very important. The Strength limit state during construction when the concrete is not fully cured may govern the design of the top flange in the positive-moment region, as specified in LRFD Article 6.10.3.4.

Bottom Flange (Tension Flange). The bottom flange, if properly proportioned, is not governed by the construction phase. The bottom flange is typically governed by the final condition. The Service II load combination permanent deformation provisions of LRFD Article 6.10.4.2 may govern and must be checked.

Negative-Moment Region Pier Section

The negative-moment region pier section will most likely be a non-compact section during all conditions. The concrete deck over the pier is in tension in the negative-moment region and, thus, considered cracked and ineffective at the nominal resistance (i.e., Strength load combinations). Thus, a good portion of the steel cross section is in compression. To qualify as compact, the web usually must be too thick to be cost effective. Thus, the cost-effective section will typically be a non-compact section.

Both top and bottom flanges in the negative moment region are typically governed by the Strength limit state in the final condition. Furthermore, the bottom flange in compression is typically governed by the location of the first intermediate diaphragm off the pier because it provides the discrete bracing for the flange.

15.6.2. Shear Connectors

Reference: LRFD Article 6.10.10

The preferred size for shear studs for use on the flanges of girders and girders is ⅞ inch diameter. Additional requirements are in the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*.

15.6.3. Stiffeners

Reference: LRFD Article 6.10.11

Transverse Intermediate Stiffeners

Reference: LRFD Article 6.10.11.1

Design straight girders without intermediate transverse stiffeners, if economical. If stiffeners are required, provide fascia girders with stiffeners only on the inside face of the web for aesthetics. Due to the labor intensity of welding stiffeners to the web, the unit cost of stiffener by weight is approximately nine times that of the unit cost of the web by weight. It is seldom economical to use the thinnest web plate permitted; therefore, investigate the use of a thicker web and fewer intermediate transverse stiffeners, or no intermediate stiffeners at all. If the design requires stiffeners, the preferred width of the stiffener is one that can be cut from commercially produced bar stock.

Weld intermediate transverse stiffeners on both sides of the compression flange. Do not weld transverse stiffeners to tension flanges. The distance between the end of the web-to-stiffener weld and the near toe of the web-to-flange fillet weld should be between $4t_w$ and $6t_w$.

The width of the projecting stiffener element, moment of inertia of the transverse stiffener, and stiffener area must satisfy the requirements of LRFD Article 6.10.11.1.

Orient transverse intermediate stiffeners normal to the web. However, where the angle of crossing is between 70 degrees and 90 degrees, skew the stiffeners so that the diaphragms or cross frames may be connected directly to the stiffeners.

Bearing Stiffeners

Reference: LRFD Article 6.10.11.2

Provide full-height bearing stiffeners for all plate girders to prevent the possibility of web buckling for both temporary and permanent supports. Provide bearing stiffeners on both web faces and at the bearing points of rolled beams and plate girders. Design the weld connecting the bearing stiffener to the web to transmit the full bearing force from the stiffener to the web due to the factored loads.

15.6.4. Deck-Overhang Cantilever Brackets

Reference: LRFD Article 6.10.3

During construction, the deck overhang brackets may induce twist in the exterior girder. The contract documents must require the contractor to check the twist of the exterior girder and bearing of the overhang bracket on the web. See Figure 15-9.

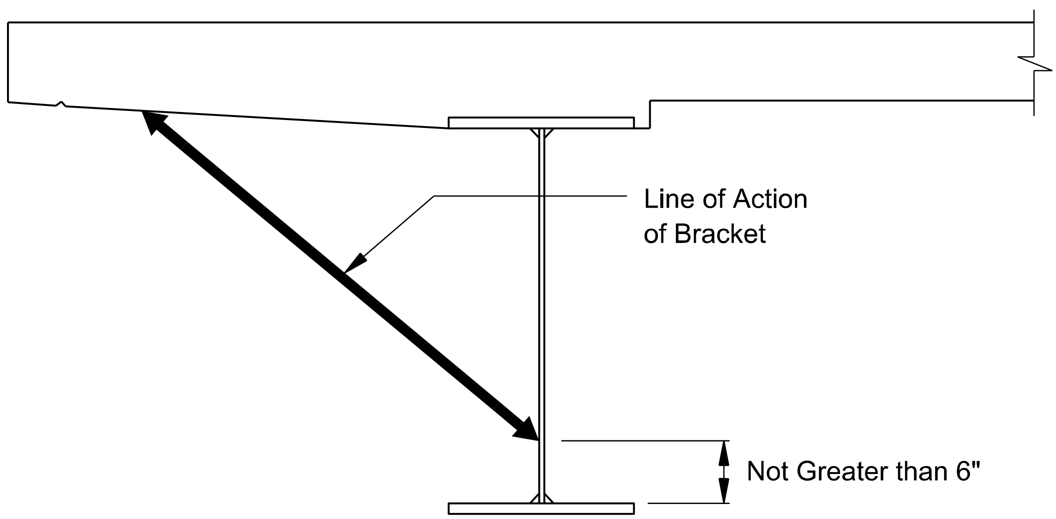

Figure 15-9
Schematic of Location for Deck Overhang Bracket

15.7. Connections and Splices

Reference: LRFD Article 6.13

15.7.1. Bolted Connections

Reference: LRFD Article 6.13.2

Design

Design all bolted connections as slip-critical at the Service II limit state, except for secondary bracing members.

15.7.2. Welded Connections

Reference: LRFD Article 6.13.3

Welding Process

The governing specification for welding new steel bridge girders is the ANSI/AASHTO/AWS *Bridge Welding Code D1.5*. However, this specification does not provide control over all of the welding issues that may arise on a project. Consult the following additional reference specifications as needed:

- AWS D1.1 for welding of tubular members and strengthening or repair of existing structures, and

- AWS D1.4 if the welding of reinforcing steel must be covered by a specification.

Include testing frequency requirements in the contract documents when not already specified by AWS or the *Alaska Standard Specifications for Highway Construction*.

Coordinate with the Chief Bridge Engineer for any issues related to the interpretation of, application for, etc., on the welding specifications. Use prequalified welds as much as possible.

Welding Types and Symbols

The primary types of welds used in bridge fabrication are fillet welds and groove welds. Welding symbols must comply with AWS A2.4 "Standard Symbols for Welding, Brazing, and Non-Destructive Examination." Welding symbols provide an instruction on the type, size, and other characteristics of the desired weld. When these symbols are properly used, the meaning is clear and unambiguous. If not used exactly as prescribed, the meaning may be ambiguous, leading to problems for all involved. The *AISC Steel Construction Manual* and most steel design textbooks have examples of welding symbols that, although technically correct, are more complicated than the typical bridge designer needs. With minor modifications, the examples in Figure 15-10 will suffice for the majority of bridge fabrication circumstances.

Field Welding

Prohibit field welding for all but a few special applications. These permissible applications are:

- welded splices for piles
- connecting pile tips to piles
- shear stud connections
- sole plates to bottom flange plates

Do not permit direct welding of stay-in-place (SIP) deck forms to girder flanges. Weld metal forms to a strap or angle that is placed over the flange.

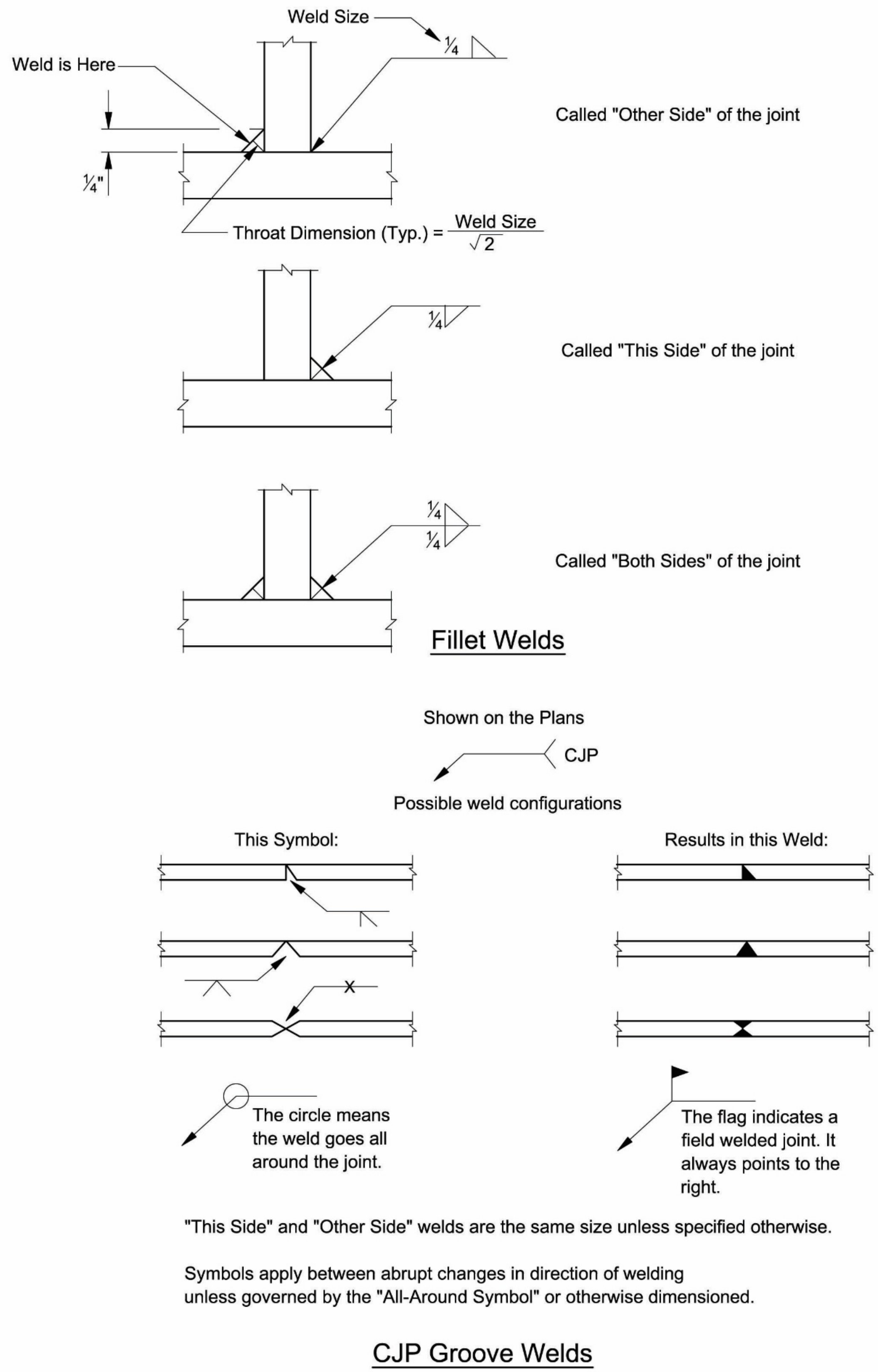

Figure 15-10
Welding Symbols

Weld-Metal Strength and Electrode Nomenclature

The strength of the weld filler metal is known from the electrode designation. Figure 15-11 illustrates the standard nomenclature to identify electrodes. The figure represents more than a bridge designer typically needs to know but, as an illustration, is informative. This applies to electrodes with a weld-metal tensile strength of 70,000 psi and the indicated welding procedures for all positions of welding or only flat and horizontal positions, respectively.

To make a weld of sufficient strength, the designer must consider three variables — weld length, weld throat, and weld-metal strength. Because weld strength is a function of these three variables, many possible combinations can yield sufficient weld strength. Full-penetration groove welds in tension require matched welds because the length and throat of the weld also match the dimensions of the base metal.

Design of Welds

The design of fillet welds is integral to LRFD Section 6 on Steel Design. The *LRFD Specifications* addresses topics such as resistance factors for welds, minimum weld size, and weld details to reduce fatigue susceptibility.

The weld-strength calculations of LRFD Section 6 assume that the strength of a welded connection is dependent only on the weld metal strength and the area of the weld. Weld metal strength is a self-defining term. The area of the weld that resists load is a product of the theoretical throat multiplied by the length. The theoretical weld throat is the minimum distance from the root of the weld to its theoretical face. See Figure 15-10.

These digits indicate the following:	
Exx1z	All positions of welding
Exx2z	Flat and horizontal positions
Exx3z	Flat welding positions only
These digits indicate the following:	
Exx10	DC, reverse polarity
Exx11	AC or DC, reverse polarity
Exx12	DC straight polarity, or AC
Exx13	AC or DC, straight polarity
Exx14	DC, either polarity or AC, iron powder
Exx15	DC, reverse polarity, low hydrogen
Exx16	AC or DC, reverse polarity, low hydrogen
Exx18	AC or DC, reverse polarity, iron powder, low hydrogen *This is most common.*
Exx20	DC, either polarity, or AC for horizontal fillet welds; and DC either polarity, or AC for flat position welding
Exx24	DC, either polarity, or AC, iron powder
Exx27	DC, straight polarity, or AC for horizontal fillet welding; and DC, either polarity, or AC for flat position welding, iron powder
Exx28	AC or DC, reverse polarity, iron powder, low hydrogen

Note: *The "xx" shown above is a two-digit number indicating the weld metal tensile strength in 1000 psi increments. For example, E7018 is 70,000 psi.*

Figure 15-11
Electrode Nomenclature

Fillet welds resist load through shear on the throat, while groove welds typically resist load through tension, compression, or shear depending upon the application.

Often, it is best to only show the type and sizes of the weld required and leave the details for the specific weld geometry to the fabricator.

When considering design options, note that the most significant factor in the cost of a weld is the volume of the weld material that is deposited. Over-sizing a welded joint is unnecessary and uneconomical. A single-pass weld is one made by laying a single weld bead in a single move of the welder along the joint. A multiple-pass weld is one in which several beads are laid one upon the other in multiple moves along the joint. Welds sized in a single pass are preferred because these are most economical and least susceptible to resultant discontinuities. The maximum weld size for a single-pass fillet weld is 5/16 inch. The AWS D1.1 *Structural Welding Code*, Table 3.7, provides more specific maximum single-pass fillet-weld sizes for various welding processes and positions of welding. Design the weld economically, but its size should not be less than ¼ inch and, in no case, less than the requirements of LRFD Article 6.13.3.4 for the thicker of the two parts joined. Show the weld terminations.

The following types of welds for girders are prohibited:

- field-welded girder splices
- intersecting welds
- intermittent fillet welds
- partial penetration groove welds

Provide careful attention to the accessibility of welded joints. Provide sufficient clearance to enable a welding rod to be placed at the joint. Often, a large-scale sketch or an isometric drawing of the joint will reveal difficulties in welding or where critical weld stresses must be investigated.

Inspection and Testing

A systematic program of inspection and testing is indispensable to the reliable use of welding. Inspection is done at the shop and at the field site. The function of the inspection is to guarantee the use of specified materials and procedures under conditions where proper welding is possible. If the sequence of welding has been specified, the inspector should be able to certify conformance.

Despite careful inspection, weld discontinuities may escape detection unless all or part of the work is subjected to tests. There are two broad categories of testing — destructive testing, which is used very sparingly for big problems or forensic studies, and nondestructive testing, which is used routinely to guarantee the quality of the welds. DOT&PF uses the following types of non-destructive evaluation (NDE):

1. **Radiographic Testing (RT)**. Used to find cracks and inclusions after a weld is completed. The process involves placing film on one side of the weld and a source of gamma or x-rays on the other side of the weld. Shadows on the exposed film indicate cracks or inclusions in the welds or adjacent areas. Use RT for groove-weld inspection. It is most effective when both sides of the weld are accessible.

2. **Ultrasonic Testing (UT)**. Relies on the reflection patterns of high-frequency sound waves, which are transmitted at an angle through the work. Cracks and defects interrupt the sound transmission, altering the display on an oscilloscope. UT can reveal many defects that the other methods do not, but it relies very heavily on the interpretative skill of the operator. Use UT for the inspection of groove welds and is preferred when a permanent record is desired.

3. **Magnetic Particle Testing (MT)**. Performed by covering the surface of a weld with a suspension of ferromagnetic particles and then applying a strong magnetic field. Cracks interrupt the magnetic force lines, causing the particles to concentrate near the crack in patterns easily interpreted by the inspector. Use MT for inspecting fillet welds.

4. **Dye Penetrant Testing (DP)**. Use a dye in liquid form to detect cracks. Capillary tension in the liquid causes the dye to penetrate into the crack, remaining behind after the surface is clean. Use DP to locate surface flaws in and around fillet welds.

5. **Visual Testing (VT)**. Required on all welds.

To aid the inspector, the contract documents for continuous structures must identify the location of tension regions along both the top and bottom girder flanges. Show the length of each stress region and reference these regions to the point of support. Figure 15-12 illustrates the information required.

15.7.3. Splices

Reference: LRFD Article 6.13.6

Shop Splices

In addition to the provision of LRFD Article 6.13.6, the following applies to shop splices:

1. **Location**. Numerous groove welds and/or groove welds located in high stress regions are not desirable. Locate flange shop splices away from high moment regions and web splices away from high shear regions. This is simple for flange splices in negative moment regions but more difficult with positive moment regions. In positive moment areas, the magnitude of moment does not change quickly along the girder compared to the negative moment.

   The location of shop groove splices is normally dependent upon the length of plate available to the fabricator. This length varies depending upon the rolling process. Most plates are available (e.g., Grades 36 and 50) in lengths up to 80 feet depending on thickness. Consider the cost of adding a shop-welded splice instead of extending a thicker plate when designing members. Discuss these issues with a fabricator or the NSBA during design.

2. **Welded Shop Splice**. Figure 15-13 illustrates welded flange splice details. At flange splices, the thinner plate should not be less than one-half the thickness of the thicker plate. See LRFD Article 6.13.6.2 for more information on splicing different thicknesses of material using groove welds.

Field Splices

In addition to the provisions of LRFD Article 6.13.6, the following applies to field splices:

1. **Welded Field Splices**. These are prohibited.

2. **Location**. In general, locate field splices in main girders at low-stress areas and near the points of dead-load contraflexure for continuous spans. Long spans may require that field splices be located in high moment areas.

3. **Bolts**. Calculate design loads for bolts by an elastic method of analysis. Provide at least two lines of bolts on each side of the web splice and four lines in flange splices.

4. **Composite Girder**. If a compositely designed girder is spliced at a section where the moment can be resisted without composite action, design the splice as non-composite. If composite action is necessary to resist the loads, design the splice for the forces due to composite action.

5. **Design**. Design bolted splices to satisfy both the slip-critical criteria under Service II loads and the bearing-type connection criteria under the appropriate Strength limit states.

6. **Swept Width (or shipping width) for Horizontally Curved Girders**. The swept width is the horizontal sweep in a curved girder plus its flange width. Locate field splices such that the maximum swept width for a horizontally curved girder is 10 feet within a single field section. Otherwise special hauling/shipping requirements apply at a substantial cost.

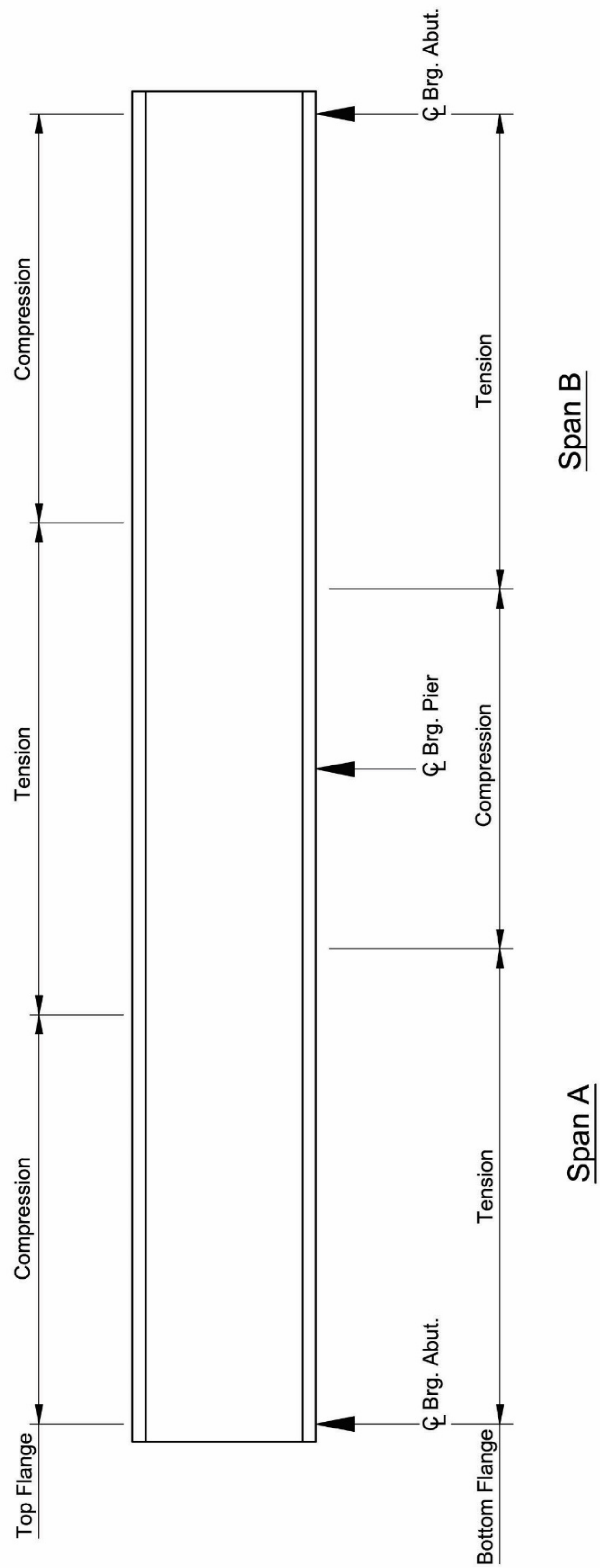

Figure 15-12
Schematic of Flange Tension Regions

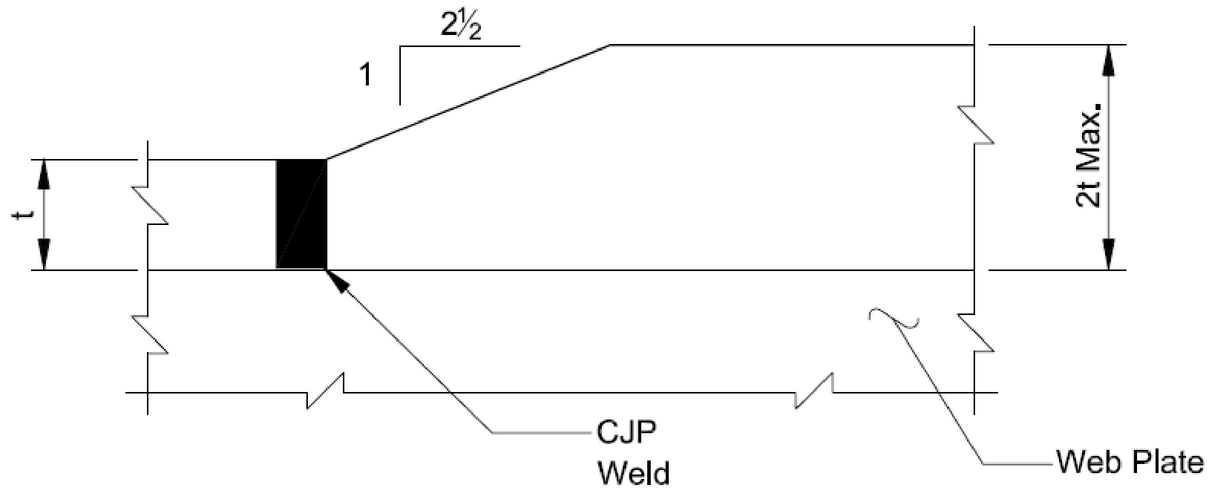

Figure 15-13
Typical Welded Splice Details

16. Bridge Decks and Rails

Sections 3, 4, and 9 of the *LRFD Bridge Design Specifications* present the AASHTO criteria for the structural design of bridge decks. Section 3 specifies loads, Section 4 specifies the modeling and analyses, and Section 9 specifies bridge deck design. Unless noted otherwise in this chapter, the *LRFD Specifications* apply to the design of bridge decks in Alaska.

This chapter documents DOT&PF criteria on the design of deck-on-girder structures; i.e., bridge decks that are constructed compositely in conjunction with concrete or steel girders. Chapter 14 discusses the design of precast, prestressed-concrete, decked bulb-tee girders.

16.1. General

16.1.1. Types/Usage

DOT&PF typically uses a 4-inch asphalt overlay with a waterproofing membra ne on all bridges (both cast-in-place and precast) on the National Highway System (NHS). The types of bridge decks most commonly used by DOT&PF and their typical usage are:

1. **Precast Deck Panels**. Typically, use these on non-NHS facilities in remote locations where concrete batching is impractical. Also, consider their use where accelerated bridge construction is warranted.
2. **Cast-in-Place Decks**. Typically, use these on NHS facilities in urban areas on steel or concrete bridges other than decked girder bridges.

16.1.2. Protection of Reinforcing Steel

Reference: LRFD Articles 2.5.2.1 and 5.14.

DOT&PF typical practice is to use epoxy-coated reinforcing steel in the bridge deck and approach slabs. Epoxy coat both layers of deck reinforcing and all reinforcing extending into the deck from precast and cast-in-place construction. Provide 2½ inches minimum concrete cover from the top surface of the deck to the top layer of reinforcing steel and a 4-inch asphalt overlay with a waterproofing membrane.

16.1.3. Traditional Design Using the "Strip Method"

Reference: LRFD Articles 9.7.3, 4.6.2.1.1, 4.6.2.1.3, and Appendix A4.

Use the traditional deck design using the "strip method" based on LRFD Articles 9.7.3 and 4.6.2.1 for deck design. Do not apply the empirical design procedures of LRFD Article 9.7.2 to the deck design.

The bridge engineer may apply the strip method to concrete decks using the design table for deck slabs in Appendix A4 of the *LRFD Specifications* (LRFD Table A4-1). The introduction to the LRFD table discusses its application and inherent assumptions.

The bridge engineer may apply the LRFD Table A4-1 to design the concrete deck reinforcement. LRFD Table A4-1 tabulates the resultant live-load moments per unit width for reinforcement design as a function of the girder or web spacings, "S." The table distinguishes between positive moments and negative moments and tabulates negative moments for various design sections as a function of the distance from the girder or web centerline to the design section. LRFD Article 4.6.2.1.6 specifies the design sections to be used.

16.2. Design Details

16.2.1. General

The following general criteria apply to bridge decks that are constructed compositely in conjunction with concrete girders and steel girders. Also comply with the requirements in Sections 14.1 and 14.2 that apply to bridge decks.

1. **Thickness**. The thickness of reinforced concrete decks is a 7-inch minimum.
2. **Reinforcing Steel Strength**. The specified yield strength of reinforcing steel must be 60 ksi.
3. **Exposure Condition**. Use a Class 2 exposure factor in LRFD Equation 5.6.7-1 for all bridge decks.
4. **Placement of Top and Bottom Transverse Reinforcing Steel**. The top and bottom transverse reinforcing steel should be offset, preferably at half the spacing. Do not place the top mat directly above the bottom mat.
5. **Reinforcing Steel Spacing**. See Table 14-3.
6. **Reinforcing Bar Size**. The minimum reinforcing steel size used for primary bridge deck reinforcement is a #5 bar. The maximum reinforcing steel size used for bridge deck reinforcement is a #6 bar.
7. **Sacrificial Wearing Surface**. The 2½-inch top reinforcement concrete cover includes ½-inch that is considered sacrificial. For both the deck and superstructure, include its weight as a dead load, but do not include its structural contribution in the structural design. Additional sacrificial concrete cover may be required in regions with heavy truck traffic and without asphalt overlays, such as the Dalton Highway.
8. **Concrete Strength**. The minimum specified 28-day compressive strength of concrete for bridge decks is 5 ksi for Class A-A concrete.
9. **Placement of Transverse Reinforcing Steel on Skewed Bridges**. The following applies:

   a. Skews ≤ 20 degrees: Place the transverse reinforcing steel parallel to the skew.

   b. Skews > 20 degrees: Place the transverse reinforcing steel perpendicular to the longitudinal reinforcement.

   See Section 16.2.5 for a definition of skew angle and for structural considerations related to skewed reinforcing steel placement.
10. **Splices**. See Section 14.2 for splicing requirements.
11. **Shear Connectors for Precast, Prestressed Concrete Bulb-Tee Girder Bridges**. Stirrups should project from the girders into the slab in accordance with LRFD Article 5.7.4 to provide a composite section. Detail bars to hook around longitudinal deck reinforcement.

16.2.2. Precast Deck Panels

Figure 16-1 presents typical connection details for precast concrete deck panels.

16.2.3. Deck Haunches

Haunches consist of concrete between the top of a steel flange or concrete girder and the soffit of the bridge deck; they account for construction variations and tolerances.

The haunch may vary across the width of the flange based on the cross slope. It may also vary along the length of the girder due to flange thickness, camber variation, and roadway profile.

In all cases, however, the design haunch thickness should be a ½-inch greater than the theoretical minimum including camber tolerances, bolted field splices, etc.

Include the girder haunch in the load calculations as dead load by applying the maximum haunch dimension throughout the span; however, ignore the haunch in the section's resistance calculations.

Measure the control dimension "Y" (see Figures 16-2 through 16-4) at the centerline of bearing. This dimension varies along the span to compensate for variations in camber and superelevation ordinate. In some cases where vertical curve corrections are small, the bridge engineer can accommodate the vertical curve ordinate in the haunch without including it in the girder.

Detail the haunch flush with the vertical edge of the top flange.

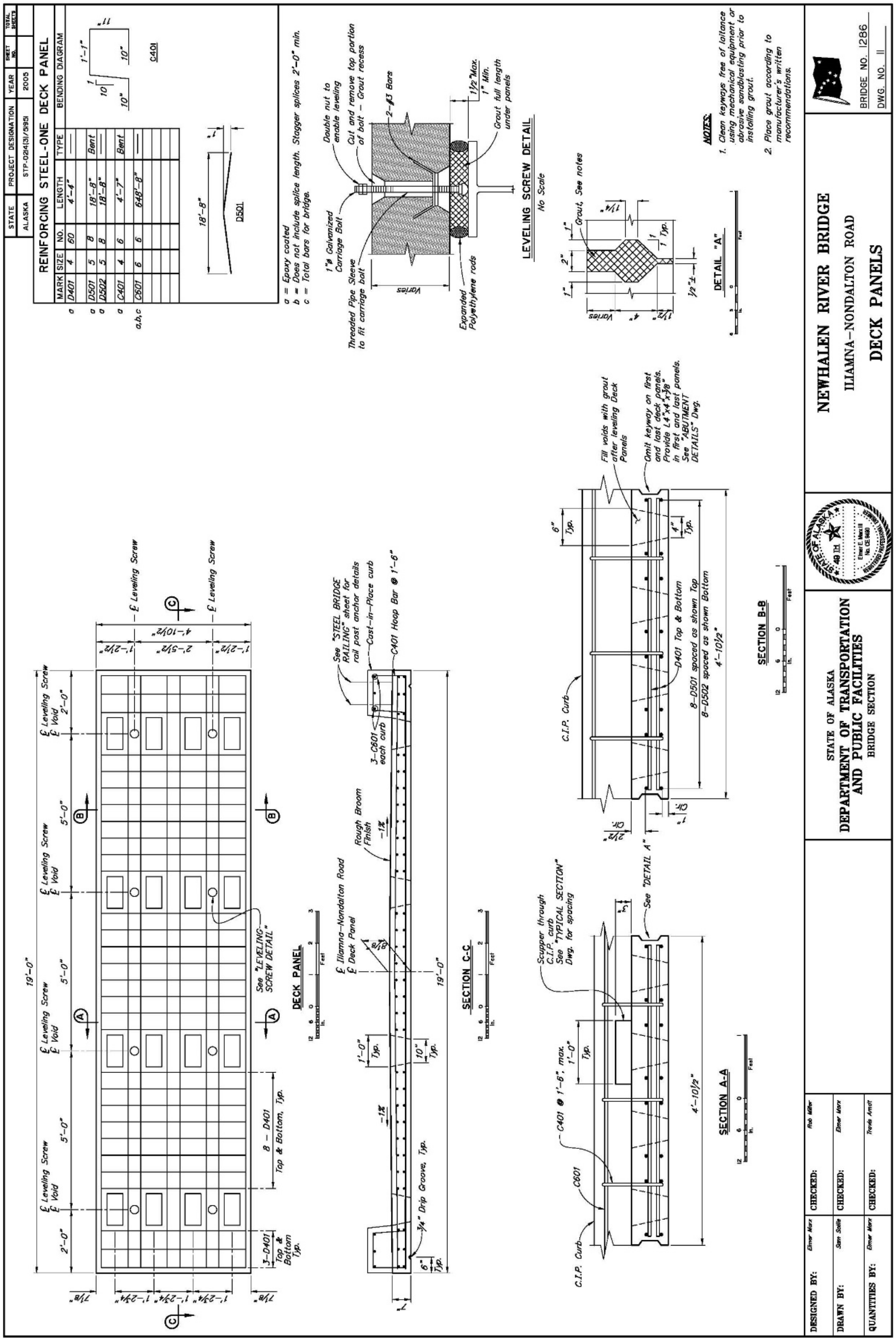

Figure 16-1
Connection Details for Precast Concrete Deck Panels

Haunch Dimensions for Steel Girders/Beams

Figure 16-2 illustrates the controlling factors used to determine the haunch dimension for steel plate girders; Figure 16-3 applies to rolled beams. Refer to the figures to determine the control dimension and haunch thickness requirements for the applicable type of steel member.

Haunch Dimensions for Precast Concrete Girders

Figure 16-4 illustrates the controlling factors used to determine the haunch dimension for precast concrete girders. Control dimension "Y" is the deck thickness "T" plus 3 inches. Use the 3-inch dimension to account for camber growth in the girder at midspan. The amount of camber growth can vary between girders cast at the same time.

Reinforcement for Deep Haunches

Provide additional reinforcement in haunches greater than 4 inches deep (X > 4 inches). The additional reinforcement should consist of a minimum of #4 U-shaped reinforcing bars spaced at a maximum of 12 inches. Properly develop these reinforcing bars into the bridge deck. See Figure 16-5.

16.2.4. Stay-in-Place Forms

Only use stay-in-place (SIP) forms to close the top of steel tub girders that are trapezoidal in shape. Use steel SIP forms.

Design loads for stay-in-place forms should consist of not less than 0.015 ksf for the metal forms and form corrugation fill applied over the areas of the forms. Do not field weld the stay-in-place forms to steel flanges. SIP forms may be shop welded to the girders.

16.2.5. Skewed Decks

Reference: LRFD Article 9.7.1.3.

Skew is defined by the angle between the centerline of support and the normal drawn to the longitudinal centerline of the bridge at that point.

The *LRFD Specifications* suggest that the effects of skew angles not exceeding 25 degrees can be neglected for concrete decks, but the *LRFD Specifications* assume the typical case of bridges with relatively large span-length-to-bridge-width ratios. Further, the commentary indicates that the 25 degrees limit is "somewhat arbitrary." Therefore, DOT&PF uses a 20 degree threshold for the consideration of skew in reinforcement detailing.

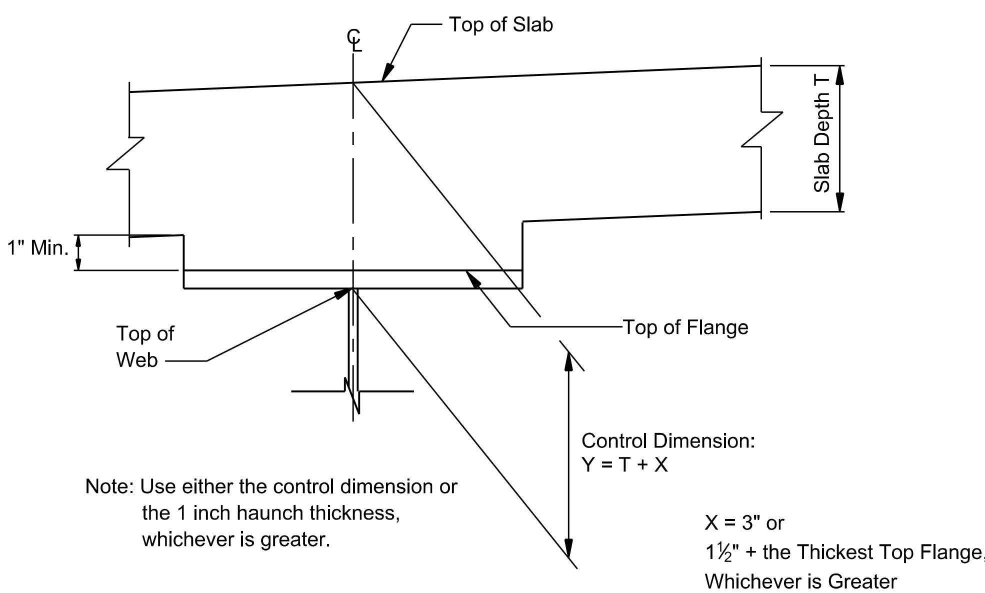

Figure 16-2
Haunch Dimension for Steel Plate Girders

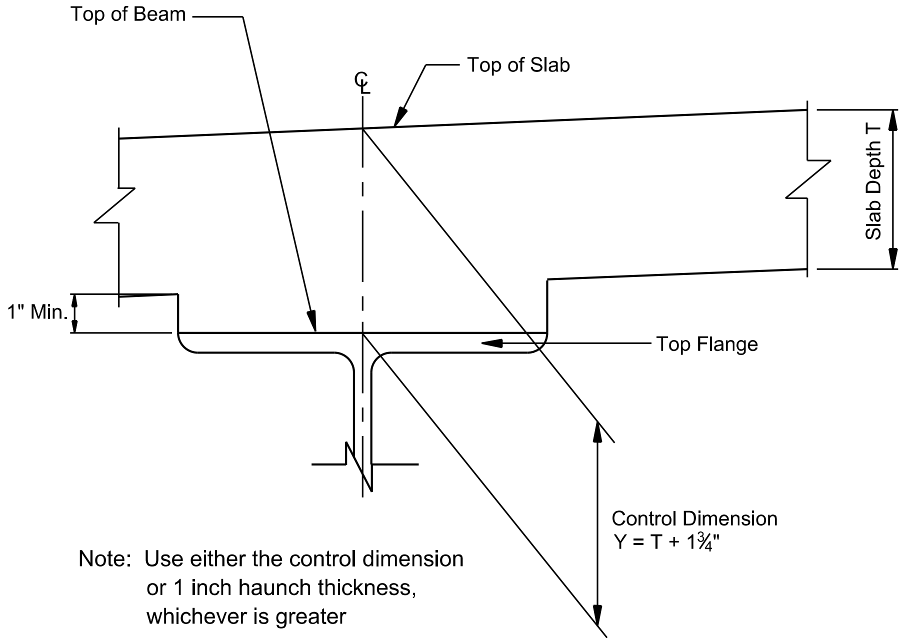

Figure 16-3
Haunch Dimension for Steel Rolled Beam

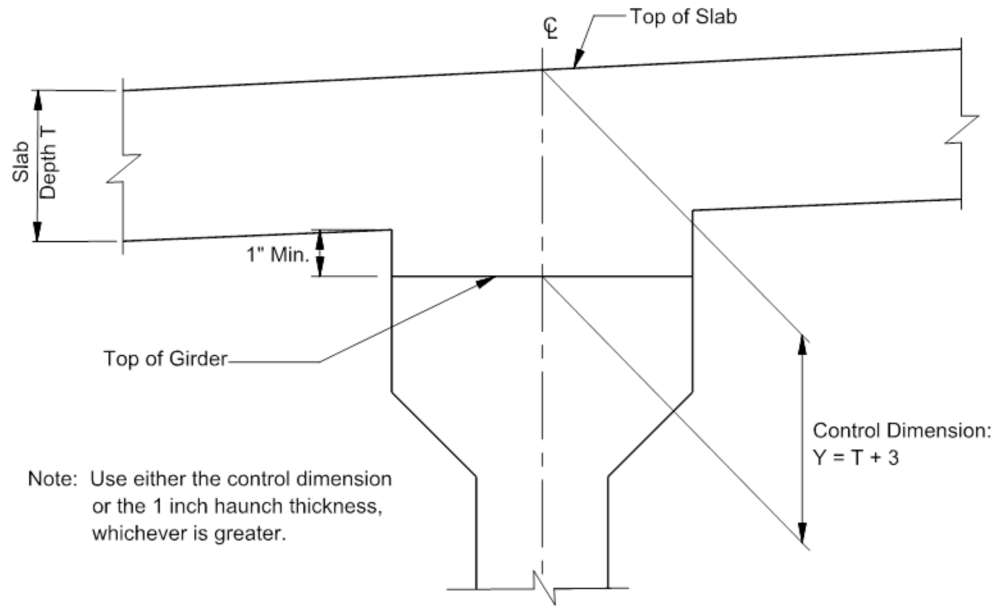

Figure 16-4
Haunch Dimension for Concrete I-Girders

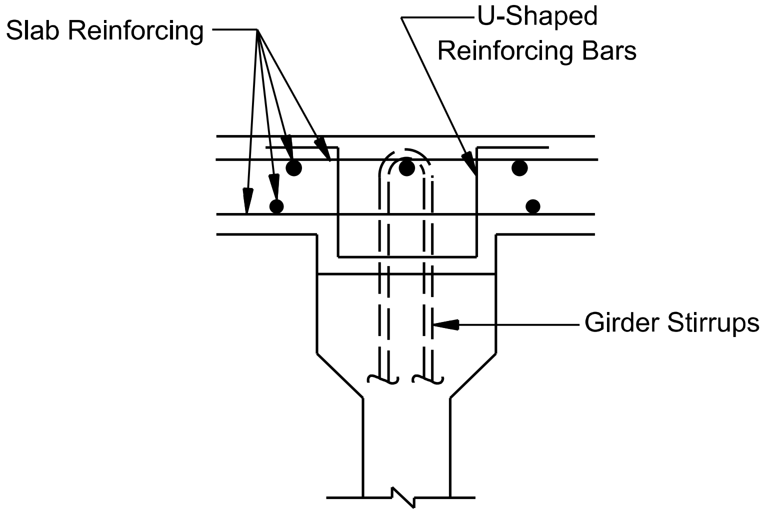

Figure 16-5
Haunch Reinforcment

16.2.6. Concrete Placing Sequence for Composite Bridge Decks

Reference: LRFD Article 2.5.3.

DOT&PF Practices

Bridge plans should include a deck placing sequence for all cast-in-place concrete decks. The deck placement diagram must present the sequence of placing concrete in various sections (separated by transverse construction joints) of deck slabs on continuous spans.

The designated sequence should avoid or minimize the dead-load tensile stresses in the slab during concrete setting to minimize cracking. Arrange the sequence to cause the least disturbance to the portions placed previously.

In addition, for longer span steel girder bridges, the placing sequence can lock-in stresses different from those associated with the instantaneous placement typically assumed in design. Therefore, for these bridges, consider the placing sequence in the design of the girders.

Deck placement should be uniform and continuous over the full width of the superstructure. The first pours should include the positive-moment regions in all spans. For all decks on a longitudinal gradient of 3 percent or greater, the direction of placement is uphill.

Figure 16-6 illustrates a sample placing sequence diagram for a continuous bridge. For precast concrete girders, the cast-in-place abutment diaphragm is cast prior to placing the deck above it. The negative-moment regions for steel girders extend between the points of beam dead load contraflexure. For precast concrete girders, use a minimum of 15 feet on each side of the center of support or 20 percent of the span length, whichever is greater.

For simple spans, it is desirable to place the entire deck in one operation.

Transverse Construction Joints

Where used, place transverse construction joints parallel to the transverse reinforcing steel. Do not place these joints over girder field splices.

Place a transverse construction joint in the end span of bridge decks on steel superstructures where uplift is a possibility during deck placement.

A bridge with a relatively short end span (60 percent or less) when compared to the adjacent interior span is most likely to produce this form of uplift. Uplift during the deck placing operation can also occur at the end supports of horizontally curved decks and in superstructures with severe skews.

If analysis using the appropriate permanent load factors of LRFD Article 3.4.1 demonstrates that uplift occurs during deck placement, require a construction joint in the end span and require placing a portion of the deck first to act as a counterweight.

16.2.7. Longitudinal Construction Joints

Longitudinal construction joints in bridge decks can create planes of weakness that can lead to maintenance problems.

In general, DOT&PF discourages the use of construction joints, although they are unavoidable under certain circumstances (e.g., widenings, phased construction).

Usage

Do not use longitudinal construction joints on decks having a constant cross section where the width is less than or equal to approximately 120 feet. For deck widths greater than 120 feet (i.e., where the finishing machine span width must exceed 120 feet), make provisions to permit placing the deck in practical widths.

Detail either a longitudinal joint or a longitudinal closure pour, preferably not less than 3 feet in width. Locate lap splices in the transverse reinforcing steel within the longitudinal closure pour. Such a joint should remain open as long as the construction schedule permits to allow transverse shrinkage of the deck concrete.

Consider the deflections of the bridge on either side of the closure pour to ensure proper transverse fit up.

Location

If a longitudinal construction joint is necessary, do not locate it underneath a wheel line. Preferably, locate a construction joint outside the girder flange and in a shoulder or median area.

If practical, longitudinal construction joints should line up with lane lines to avoid driver confusion at night.

Closure Pours

For staged construction projects, use a closure pour to connect the slab between stages. A closure pour serves two useful purposes. It defers final connection of the stages until after the deflection from deck slab weight has occurred. A closure pour also provides the width needed to make a smooth transition between differences in final grades that result from construction tolerances.

The closure width should relate to the amount of relative dead-load deflection that is expected to occur across the pour after the closure is placed. A minimum closure width of 3 feet is recommended.

Greater closure widths may be required when larger relative dead-load deflections are anticipated. Estimate the required width by considering the closure pour to be a fixed-fixed beam and by limiting the stresses in the concrete to the cracking stress. When a closure pour is used, the following apply:

1. Do not rigidly connect diaphragms/cross frames in the staging bay of structural steel girders until after the adjacent stages of the deck have been placed. Construct concrete diaphragms in the staging bay of prestressed concrete girders after adjacent portions of the bridge are complete. Pour the diaphragms as part of the closure.
2. Reinforcing steel between different stages must not be tied or coupled until after the adjacent stages of the deck have been placed.
3. Support the finishing machine on an overhang jack that is connected to the girder loaded by the deck pour. Do not place the finishing machine on a previously poured deck. Indicate in the contract documents that this method of constructing the closure pour is not allowed. See Figure 16-7.

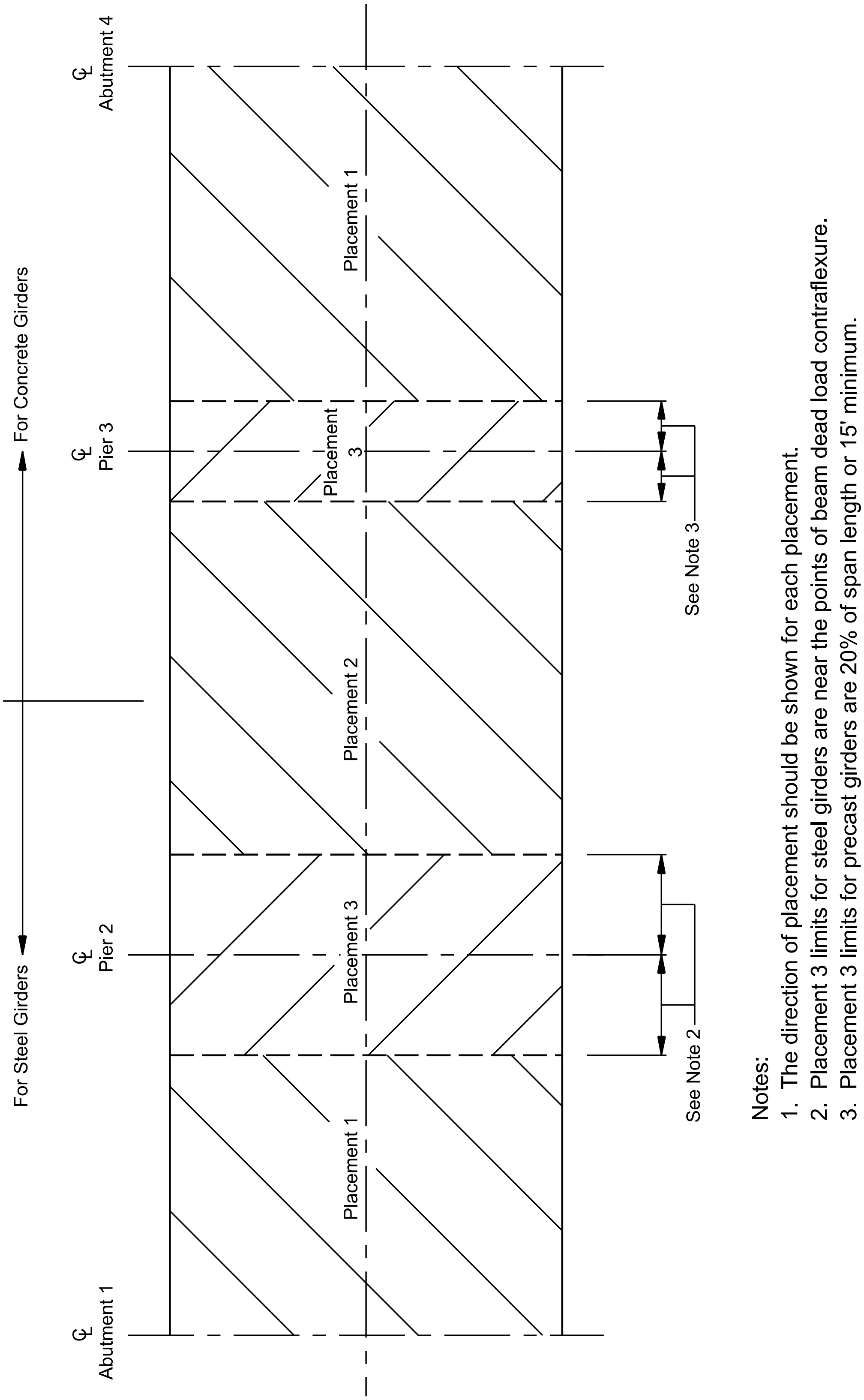

Figure 16-6
Typical Placement Diagram
(Continuous Steel and Precast Girders)

16.2.8. *Transverse Edge Beam for Steel Girder Bridges*

Reference: LRFD Article 9.7.1.4.

DOT&PF practice is to provide a transverse edge beam to support wheel loads near the transverse edge of the deck in conjunction with an end diaphragm for steel girder bridges. See Figure 16-8.

16.2.9. *Deck Overhang/Bridge Rail*

Reference: LRFD Article 9.7.1.5.

Overhang Width and Thickness

Bridge deck overhang is defined as the distance between the centerline of the exterior girder to the outside edge of the deck.

DOT&PF practice is that the overhang width will not be more than 40 percent of the girder spacing for I-girders and 50 percent for box girders.

The thickness of the overhang at the outside edge of deck may be less than the interior deck thickness but not less than 7 inches.

The thickness of the overhang at the outside edge of girder should be the deck thickness plus the haunch depth.

Construction

Typical DOT&PF practice is to construct the exterior overhang of the cast-in-place bridge deck slab using an overhang jack for steel and precast concrete girders.

Overhang jacks are connected to the girder at their top and braced against the web or bottom flange on the bottom.

Large overhang widths can cause excessive lateral distortion of the bottom flange and web of the girder. The Contractor must check the twist of the exterior girder and bearing of the overhang bracket on the web.

See Figure 16-9 for typical overhang construction forming on concrete and steel girders.

Structural/Performance Design of Bridge Railing

Reference: LRFD Articles 13.6.1, 13.6.2, and 13.7.2.

All combination bridge rail/deck overhang designs must meet the structural design requirements to sustain rail collision forces in LRFD Article A13.2.

With the Alaska Multi-State Rail or the Oregon Three-Tube Rail, the railing requirements in Article 13.6 are considered satisfied if the appropriate bars shown in Table 14-4 are used, or if the deck moment resistance provided is greater than or equal to the moment resistance of the applicable overhang shown in Table 14-4.

Use a Class 2 exposure factor in LRFD Equation 5.6.7-1 for all bridge rails and deck overhang designs.

Rail Joints

Provide joints on concrete bridge rails and curbs at all locations of expansion in the bridge; i.e., the joints on the bridge deck and barrier will match. Consider additional open joints on longer spans. Design open joints as discontinuities.

The expansion joint seal or hardware should extend up into the barrier or curb at least 12 inches.

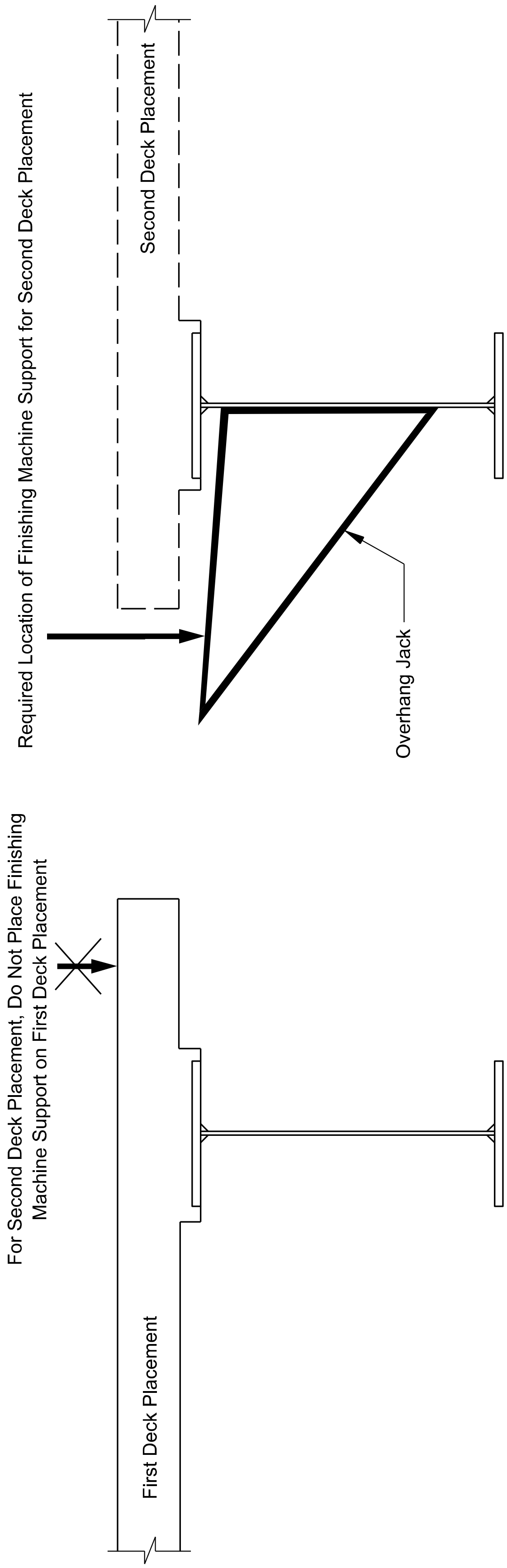

Figure 16-7
Support for Finishing Machine

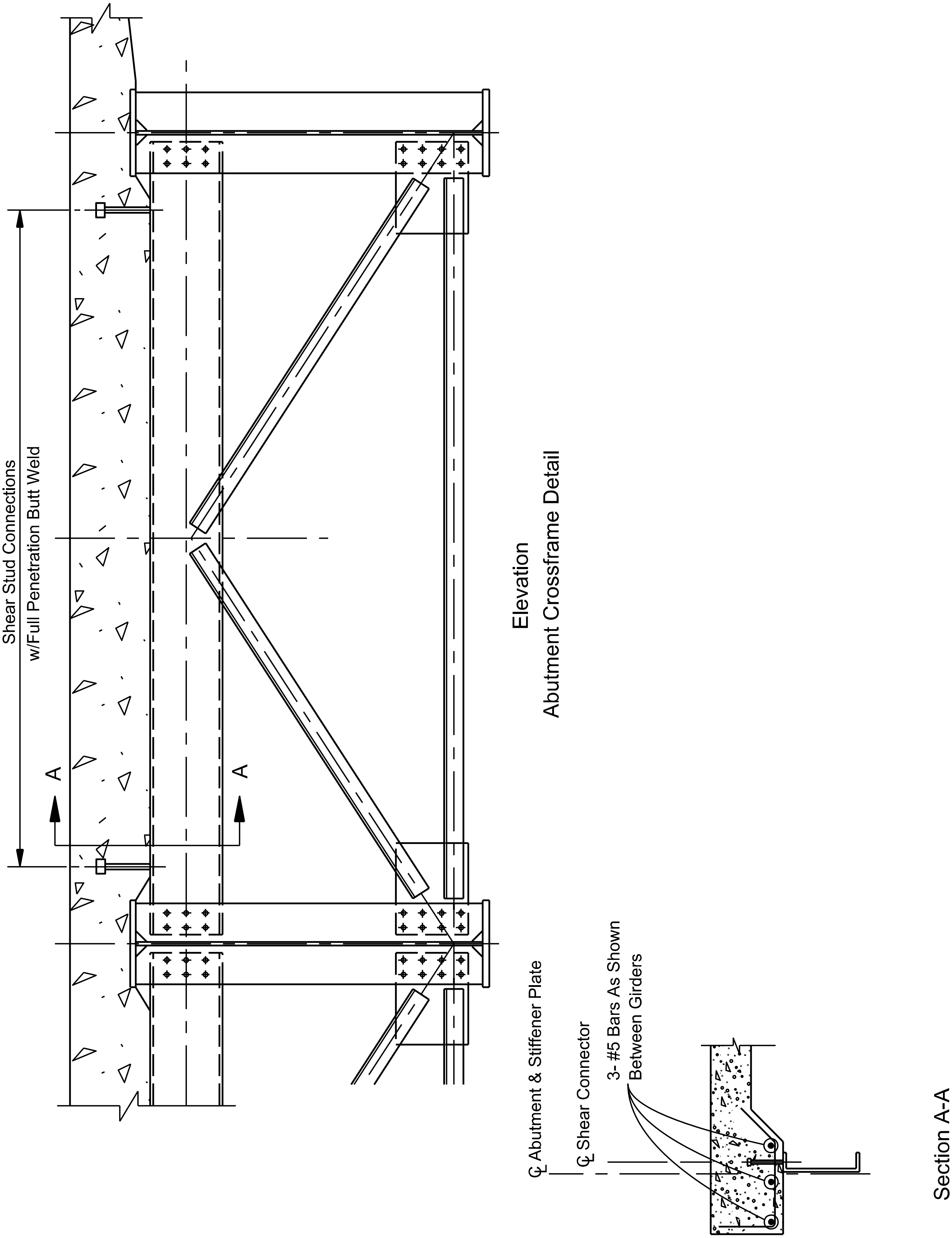

Figure 16-8
Transverse Edge Beam

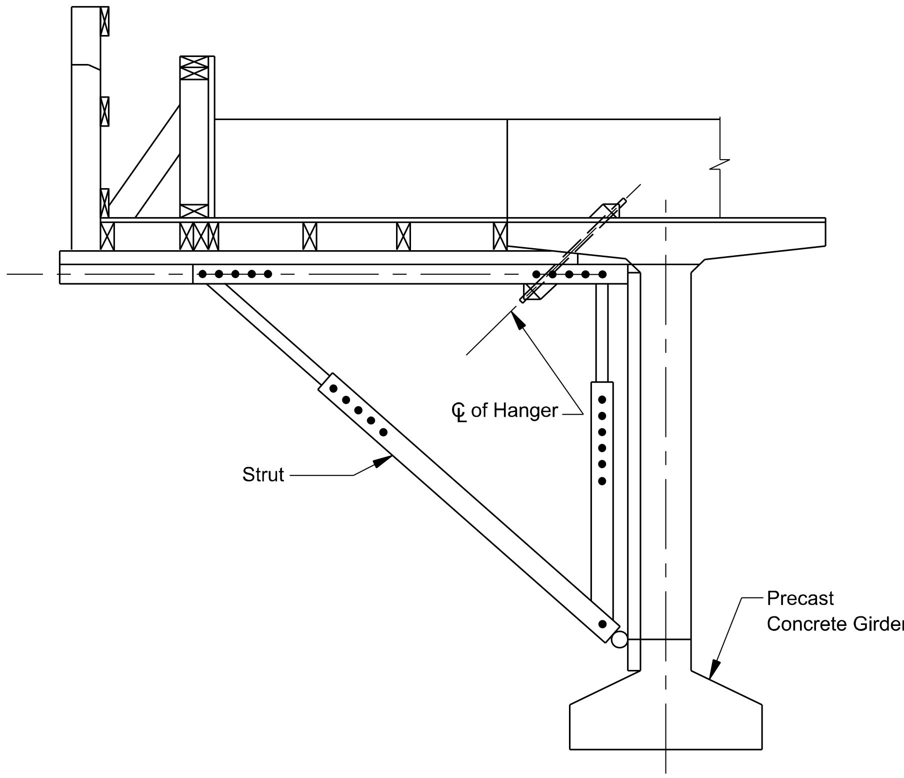

Figure 16-9
Typical Overhang Forming System
(Concrete and Steel Girders)

16.3. Approach Slabs

16.3.1. Usage

Approach slabs are required on all bridges where a concrete batch plant is within a one-hour drive to the bridge site and on bridges with cast-in-place decks.

16.3.2. Design Criteria

The following design criteria apply to approach slabs:

1. **Materials**. Use concrete with a minimum f'_c of 4 ksi. The class of concrete used in the approach slabs should be consistent with the class used in the deck to which they are attached, but may be Class A concrete for decked bulb-tee girder bridges. All reinforcing steel in the approach slab must be epoxy-coated.
2. **Analysis**. If a special design is used, model the approach slab as a simple span.
3. **Slab Length**. The minimum approach slab length should be the length of the wingwall or 15 feet, whichever is greater. Design approach slabs longer than 20 feet as a longitudinally reinforced slab span. The design should assume that the approach slab is a simple span supported by the bridge on one end and by approximately 3 feet of competent soil at the approach roadway end. Use the Extreme Event II load combination.
4. **Skew**. Design the roadway ends of approach slabs parallel to the bridge ends except where the skew is more than 20 degrees. For bridge skews greater than 20 degrees, design the angle of the roadway end of the approach slab to avoid interference with snow plow equipment.

16.4. Deck Drainage

Reference: LRFD Article 2.6.6.

16.4.1. Importance of Bridge Deck Drainage

The bridge deck drainage system includes the bridge deck, sidewalks, railings, gutters, and inlets. The primary objective of the drainage system is to remove runoff from the bridge deck before it collects in the gutter to a point that exceeds the allowable design spread (typically, runoff water must be restricted to the shoulder portion of the deck). Proper bridge deck drainage provides many other benefits, including:

- efficiently removing water from the bridge deck to enhance public safety by decreasing the risk of hydroplaning;
- enhancing long-term maintenance of the bridge;
- preserving the structural integrity of the bridge;
- enhancing aesthetics (e.g., the avoidance of staining substructure and superstructure members); and
- reducing erosion on bridge end slopes.

See Section 10.6.10 of the *Alaska Highway Drainage Manual* for DOT&PF policies on deck drainage.

16.4.2. Responsibilities

The Hydraulics Unit (Statewide Hydraulics Engineer):

- calculates the flow of water on the deck based on the design frequency,
- selects the type of deck drain, and
- determines the hydraulic inlet spacing on the bridge deck to intercept the calculated flow to meet the allowable water spread criteria.

The bridge engineer incorporates the drainage design information into the structural design of the bridge. The roadway designer is responsible for the drainage design for any runoff approaching or leaving the bridge deck. The bridge engineer should assist the roadway designer to ensure that the bridge end drainage is adequately addressed in the contract documents.

16.4.3. Deck Drainage Design Elements

Type of Drainage System

DOT&PF generally uses an open deck drain system.

Deck Slope

To provide proper bridge deck drainage, the minimum profile grade of the bridge should be 0.5 percent. A crest vertical curve may also be used provided the average profile grade from the high point to the ends of the bridge is greater than 0.5 percent.

Accommodate the transverse slope of the bridge deck by providing a suitable roadway cross slope, typically 2 percent for paved roads and 3 percent for gravel roads.

Sag Vertical Curves

If practical, do not locate any portion of a bridge in a sag vertical curve. If the bridge is located in a sag, do not locate the low point of the sag on the bridge or the approach slab. Locate the low point a minimum of 20 feet from the end of the approach slab or, if approach slabs are not used, a minimum of 20 feet from the end of the bridge.

Inlets/Downspouts

The bridge engineer should consider the following when locating inlets and downspouts:

1. **Location With Respect To Structural Elements**. Extend downspouts below structural elements as specified in the *LRFD Specifications*. Do not locate downspouts within 5 feet of the end of any substructure unit or where water could easily blow over and run down a substructure element.
2. **Location With Respect To Ground**. A free fall exceeding 25 feet will sufficiently disperse the falling water so that minimal erosion damage will occur beneath the bridge.

   Where less than 25 feet of free fall is available, consider providing erosion protection on natural ground beneath the outlet. Free falls of less than 25 feet are acceptable where the water free falls onto riprap or flowing water.
3. **Railroads**. Do not allow downspouts over Railroad right-of-way without the Railroad's consent.
4. **Other Exclusions**. Avoid locating downspouts over the traveled way portion of an underpassing highway, sidewalk, or unpaved embankment.

Structural Considerations

The primary structural considerations in drainage system design are:

1. **Deck Reinforcement**. Inlet sizing and placement must be compatible with the structural reinforcement and other components of a bridge deck.

   Where required, provide a thickened deck and additional reinforcement to maintain clearances and deck resistance.

2. **Corrosion and Erosion**. Design the drainage system to deter runoff (and the associated corrosives) from contacting vulnerable structural members and to minimize the potential for eroding embankments. To avoid corrosion and erosion, the design must include the proper placement of outfalls.

   To prevent erosion, direct running water away from the end of wingwalls to the end of a bridge.

Maintenance Considerations

The drainage system will not function properly if clogged with debris or ice. Therefore, it is important to consider maintenance requirements in the design. Provide easy access, adequate space, and safe working conditions for maintenance personnel to maintain the drainage features around the bridge.

Bridge End Drainage

The roadway designer is responsible for designing the bridge end drainage. The typical DOT&PF practice is to use a riprap-lined ditch (drainage swale).

In addition, design bridge end drainage with grate inlets, curb opening inlets, or combination inlets. Design bridge deck inlets at the downslope end of the bridge to collect all of the flow not intercepted by any other bridge deck inlets.

At bridge ends where the approach roadway has curb and gutter, provide catch basins as close as practical to the approach slabs.

16.5. Bridge Deck Appurtenances

16.5.1. Bridge Rails

Reference: LRFD Article 13.7.

Test Levels

LRFD Article 13.7.2 identifies six test levels for bridge rails, adopted from NCHRP 350 *Recommended Procedures for the Safety Performance Evaluation of Highway Features* and the AASHTO *Manual for Assessing Safety Hardware* (MASH). Test Levels One and Two (TL-1 and TL-2) are typically used in work zones in Alaska. Most bridge rails used by the DOT&PF meet the performance criteria for Test Levels Three or Four (TL-3 or TL-4). Only use bridge rails that meet Test Levels Five and Six (TL-5 and TL-6) criteria with the approval of the Chief Bridge Engineer.

Bridge Rail Types/Usage

For new bridge construction, select a bridge rail according to Figure 16-10. The following are preapproved TL-4 Bridge Rails for use on NHS and other State-owned arterial or collector highways:

1. **Alaska Multi-State Rail**. This bridge rail is the most commonly used type on new and replaced bridge rail installations. The Alaska Multi-State Rail meets the TL-4 performance criteria and has a lower dead weight than concrete bridge barriers. In most cases, this rail is easily adaptable to the bridge types used by the DOT&PF.

2. **32-inch Concrete F-Shape Bridge Barrier**. DOT&PF typically uses this barrier on bridges where the Alaska Multi-State Rail is not appropriate. The 32-inch, F-shape bridge rail meets the performance criteria for a TL-4. The concrete barrier has relatively low maintenance cost.

These two types represent the vast majority of bridge rails installed by DOT&PF. If the regional office desires another railing option, other available TL-4 bridge rail types include:

- 42-inch concrete Type F-shape bridge barrier,
- 42-inch three-tube combination rail (Oregon three-tube),
- 32-inch or 42-inch vertical concrete wall, or
- other metal beam bridge rails.

Guardrail-To-Bridge-Rail Transitions

The bridge engineer is responsible for the guardrail-to-bridge-rail transition. The Bridge Section has developed standard details used for most applications (see Alaska Standard Drawings G30 (TL-3) and G31 (TL-4)). The bridge engineer develops special designs, if required, for unusual circumstances.

Bridge Rail/Sidewalk

Reference: LRFD Articles 13.4 and 13.7.1.1.

The roadway designer or regional traffic engineer determines the warrants for a sidewalk on a bridge. Sidewalks on bridges create several issues that the bridge engineer must address, including:

- They complicate the bridge rail and guardrail-to-bridge-rail transition designs.
- They require special attention to bridge deck drainage.

Do not provide a sidewalk on a bridge if none exists on the approaching roadway.

16.5.2. Bicycle/Pedestrian Rails

Reference: LRFD Article 13.9.

The roadway designer or regional traffic engineer determines if bicycle accommodation is required across a bridge. Where required, provide a bicycle/pedestrian rail that meets the geometric and loading requirements of LRFD Article 13.9. The minimum required height of the bicycle rail is 42 inches.

16.5.3. Protective Fencing

Use protective fencing across bridges when protection to facilities adjacent to or beneath the structure is warranted. Fencing may be required for overpasses in urban areas and for railroad overheads. Use protective fencing at other locations if requested by the regional office. Detail gates for bridge inspection and maintenance access.

16.5.4. Utility Attachments

The Bridge Section will coordinate with the Utilities Section for any utility attachments proposed on the bridge. Refer to the Alaska Administrative Code (AAC), 17 AAC 15.231, which lists general guidelines for utility installations on bridges. The following discussion presents DOT&PF supplemental information.

Utility companies frequently request approval from DOT&PF to attach utility lines or pipes to bridges.

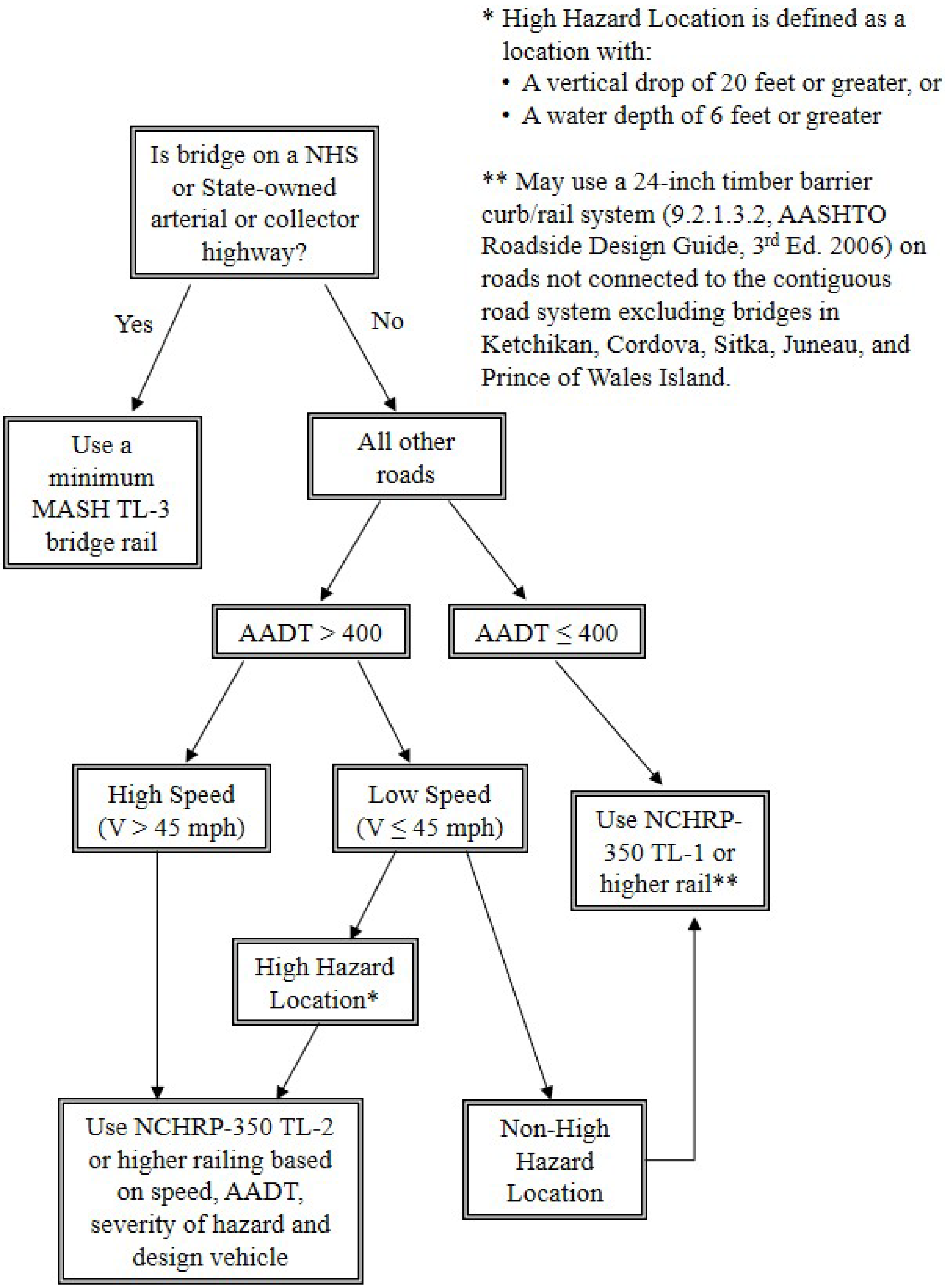

Figure 16-10
Decision Tree for Bridge Rails for New Bridge Construction

The Bridge Section is responsible for ensuring that the structural performance and function of the bridge is not compromised; that the safety of the individuals using the bridge is not compromised; and that DOT&PF bridge maintenance is not unduly complicated.

On new bridges in urban areas with no planned utilities, detail two 8-inch diameter conduits in each exterior bay of the abutment backwall and intermediate diaphragm for future use by DOT&PF or utility companies. For bridges with approach slabs, extend a utiliduct past the end of the approach slab.

DOT&PF Requirements

These requirements apply to attaching utilities to DOT&PF-owned structures.

Before proposing a utility installation to the bridge, analyze directional drilling/boring and alternative routing.

DOT&PF attempts to provide at least a 75-year service life for each structure; therefore, any utility installation should be capable of performing for a comparable service period without substantive maintenance.

The installation must be of substantial design, proportioned to span between supports without undue deflection under its own weight and the other imposed loads. The installation must be capable of accommodating the thermal expansion and contraction of the bridge.

The installation must be located between girders. Utilities 12 inch or larger in diameter must be located in non-adjacent girder bays for load distribution.

All tie rods at both bridge approach embankments must remain undisturbed.

All exposed components of the installation must be constructed of corrosion-resistant materials or have corrosion-resistant coatings.

All hanger bolts must have double nuts, burred bolt threads, or an approved thread-locking system.

The elevation of all components of the installation must be at least 1 inch above the bottom flange of the girders.

Prevent the abutment fill and/or water from spilling through the holes by filling or blocking the annular gap (created by coring or sawing holes through the abutment backwalls) between the backwalls and the utility.

Locate, map, and minimize cutting of the reinforcing steel in concrete backwalls and diaphragms. Treat all exposed reinforcing steel ends with zinc-rich paint before filling the annular gap.

Repaint any damaged areas of the bridge spray metalizing system caused by the installation of the utility, including drilled/cored holes and incidental damage. Comply with Sections 513 and 708 of the Alaska *Standard Specifications for Highway Construction* that apply to field painting of existing structures. In addition, clean the damaged area to bare metal to meet SSPC-SP 11, Power Tool Cleaning to Bare Metal.

Mark the utility owner’s name and local phone number at both abutments to allow immediate contact in an emergency.

Provide photographs of the completed installation that include typical hanger systems, general view of the utility attachment, view across piers, and photos of the utility at each abutment.

DOT&PF Prohibitions

DOT&PF does not generally permit the following:

- attachments to the underside of cast-in-place concrete decks
- attachments to bottom flanges and webs of concrete decked bulb-tees
- attachments to bridge rails or bridge rail posts
- timber utility components
- welding to or drilling holes in steel bridges

Hazardous Materials

Conduits carrying flammable, hazardous, or corrosive material are permitted on the bridge only after all reasonable crossing alternatives have been exhausted. Shutoff valves must be located beyond both abutments, outside the limits of the bridge. Keep valves operational and accessible, clear of snow, ice, dirt, and debris.

Use valve vaults where environmental elements and/or vandalism pose concern.

Provide a casing extended the full length of the bridge. Size the casing to carry the entire contents of the conduit and vent the line at points well away from the structure. Expect additional requirements on large or high-pressure lines.

Utility Company Responsibilities

A professional engineer, licensed by the state of Alaska, must design the installation. The design engineer must have design experience applicable to the proposed installation.

Provide the Bridge Section with design calculations and plans (drawn to scale) of the proposed layout, including typical sections at the abutments, attachment to the girders, and sections through the diaphragms and over the piers.

Specify the utility's pipe size, pipe thickness, insulation thickness, casing size, and casing thickness on the plan sheets.

Provide a copy of the applicable code references for the hanger/support spacing.

Provide the system's total weight per linear foot, including the weight of the contents inside the proposed conduit.

Submit the plans at least 90 calendar days before the proposed installation date.

The utility owner is responsible for removing all utility components and hangers once the service is no longer needed.

Repair damage to structural steel coatings, block and seal holes through abutment backwalls, replace riprap and structural fill disturbed by the removal, and perform any other repair deemed necessary by the Department.

Bridge plans may be available to assist in the design. Large diameter utilities may require the utility designer to provide calculations for a load rating of the bridge with the new utility loading.

Apply for a utility permit to the DOT&PF regional utility section.

16.5.5. Sign Attachments

If the Traffic Engineering Section proposes to attach a sign to a bridge, they must coordinate with the Bridge Section, which will assess the structural impact on the bridge. If approved, the Bridge Section will design the sign attachment details. Signs cannot decrease the vertical clearance.

16.5.6. Luminaire/Traffic Signal Attachments

The Traffic Engineering Section determines the warrants for highway lighting and traffic signals, and they perform the design work to determine, for example, the spacing of the luminaires and the provision of electricity.

Lighting may be included on bridges that are located in urban areas; traffic signal warrants are determined on a case-by-case basis.

Where attached to a bridge, the Bridge Section will design the structural support details for the luminaire and traffic signal attachments to the bridge.

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17. Foundations

17.1. General

A critical consideration for the satisfactory performance of any structure is the proper selection and design of a foundation that will provide adequate support. This chapter discusses Alaska-specific criteria that supplement Section 10 of the *LRFD Specifications* for the design of spread footings, driven piles, and drilled shafts. Although primarily focused on bridge foundations, this chapter addresses other transportation structure foundations.

Section 11.7 of this *Manual* presents DOT&PF criteria for selecting an appropriate foundation type within the context of structure-type selection. The *Alaska Geotechnical Procedures Manual* discusses the geotechnical considerations for bridge and transportation structure foundation design.

17.1.1. Design Methodology

The following summarizes the concepts in the *LRFD Specifications* for the design of foundations for bridges and structures.

Considering basic design principles for foundations, the *LRFD Specifications* implemented a major change when compared to the traditional principles of the AASHTO *Standard Specifications for Highway Bridges (Standard Specifications)*. The *LRFD Specifications* distinguishes between the strength of the in-situ materials (soils and rock strata) supporting the bridge and the strength of the structural components transmitting force effects to these materials. The *LRFD Specifications* emphasize the distinction by addressing in-situ materials in Section 10 "Foundations" and structural components in Sections 5 and 6, which specify requirements for concrete and steel elements. The structural engineer applies the appropriate provisions from these sections in the structural design of footings, steel and concrete piles, and drilled shafts.

Historically, the primary cause of bridge collapse has been the scouring of in-situ materials. Accordingly, the *LRFD Specifications* contain a variety of strict provisions for scour protection, which may result in deeper foundations.

17.1.2. LRFD Resistance Factors for Foundations

LRFD Article 10.5.5.1 presents resistance factors for the Service limit states, which are typically 1.0. LRFD Articles 10.5.5.2.2, 10.5.5.2.3, and 10.5.5.2.4, present resistance factors for the Strength limit state for spread footings, driven piles, and drilled shafts respectively.

17.1.3. Arctic Engineering

Some areas of Alaska have ground conditions that include permafrost, which is ground that has remained at or below 32°F for two or more years. The active layer is the surficial layer of soil that undergoes seasonal freeze/thaw cycles.

Two methods are usually recommended for installing deep foundations in permafrost. For warm permafrost (31°F), drive piles through the permafrost layer. For cold permafrost (less than 27°F), cut or drill a hole, set the pile into the hole, and then fill the annulus with slurry. The project-specific installation method will be identified in the Structural Foundation Engineering Report (SFER).

After installation, isolate piles from the soils in the active layer. DOT&PF prohibits thermal piles.

For seismic design, consider both the thawed and frozen conditions.

17.1.4. Differential Settlement

Reference: LRFD Articles 3.12.6, 10.6.2.2, and 10.7.2.3.

Differential settlement is the difference between the settlements of two adjacent foundations. In the *LRFD Specifications,* differential settlement (SE) is a superstructure load.

DOT&PF Practice

Generally, due to the methods used by DOT&PF to proportion foundations, settlements are within a tolerable range and, therefore, force effects due to differential settlement need not be investigated.

The general DOT&PF practices on the acceptable limits for settlement are:

1. **Estimated Differential Settlement**. If Statewide Materials estimates that the differential settlement is less than one-half of the total estimated settlement, the bridge engineer may usually ignore the effects of differential settlement in the structural design of the bridge.

2. **Angular Distortion**. Angular distortion is the differential settlement divided by the distance between the adjacent foundations.

   LRFD Article C10.5.2.2 states that angular distortions between adjacent foundations greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not be ordinarily permitted, and the Article suggests that other considerations may govern.

   DOT&PF does not use the LRFD limits for design, which are related to structural distress, because these angular distortions yield unacceptable impacts on ride-ability and aesthetics. Typically, meeting the requirements of Item 1. Estimated Differential Settlement should preclude exceeding the angular distortions allowed by the *LRFD Specifications*.

3. **Piers**. Consider deep foundations where differential settlement is a concern between columns within a pier.

Foundation Settlement Effects

If varying site conditions exist, the Final SFER will address settlement. Consider the following effects:

1. **Structural**. The differential settlement of substructures causes the development of force effects in continuous superstructures. These force effects are directly proportional to structure depth and inversely proportional to span length, indicating a preference for shallow, long-span structures.

   The force effects from settlement are normally smaller than expected and tend to be reduced in the inelastic phase. Nevertheless, these force effects may be considered in design if deemed significant, especially those negative movements that may either cause or enlarge existing cracking in concrete deck slabs.

2. **Joint Movements**. A change in bridge geometry due to settlement causes movement in deck joints that should be considered in joint detailing, especially for deep superstructures.

3. **Profile Distortion**. Excessive differential settlement may cause a distortion of the roadway profile that may be undesirable for vehicles traveling at high speed.

4. **Appearance**. Viewing excessive differential settlement may create a perception of lack of safety.

Foundation Settlement Mitigation

Use ground modification techniques to improve the soil to address differential settlement concerns. Some available techniques include:

- chemical grouting
- over-excavation and replacement
- surcharging
- installation of stone columns
- compaction grouting
- deep dynamic compaction

17.2. Structural Foundation Engineering Report

Use the following procedures to assist the foundation engineer in developing the Structural Foundation Engineering Report (SFER) and to provide support for bridge design and construction activities.

17.2.1. Overview and Objectives

The Department designs and constructs bridges in conformance with the *AASHTO LRFD Bridge Design Specifications (AASHTO)* and other DOT&PF documents. These specifications provide requirements for field exploration, foundation analysis, and field monitoring of bridge foundations, and aid in the development of the SFER.

SFER development requires coordination among several functional groups. The objective of this section is to:

- outline the interaction among the design team members,
- define the process for developing and implementing the SFER recommendations, and
- describe the support activities commonly required during bridge construction projects.

17.2.2. Bridge Foundation Design Process

The regional project manager (PM) requests support from the Statewide Materials Section (Geotech) to aid in developing site selection and roadway alignment options during the Preliminary Design Phase (pre-environmental document).

The PM requests support from the Bridge Section (Bridge) to develop bridge type alternatives. The PM uses Geotech and Bridge recommendations to support the identification of a preferred project alternative. Based upon the project objectives, the PM determines the preferred bridge alternative and site selection. Once selected, Bridge will send the preferred bridge alternative and site selection information (including preliminary plans in AutoCAD format) to the PM. The PM will arrange for a geotechnical investigation and foundation design recommendations by Geotech.

Note: *Geotech and Bridge typically communicate directly with each other. However, the PM is the primary contact and should be copied in most correspondence, especially matters addressing project scope, schedule, or budget. Comply with all of the requirements of the Alaska Highway Preconstruction Manual (e.g., Section 450.9.1 "Bridge Design" and Section 450.9.6 "Geotechnical Investigations").*

Geotech prepares a subsurface exploration plan based on the preferred bridge alternative(s). This typically occurs during the Preliminary Design Phase. Bridge reviews and comments on the plan. PM approval is required prior to executing the subsurface exploration plan.

Geotech and Bridge use the subsurface exploration findings to generate the Foundation Geology Report (refer to the *Alaska Geotechnical Procedures Manual* for additional information). This report helps to generate the Preliminary SFER and the Final SFER.

Geotech prepares the Preliminary SFER during the Preliminary Design Phase to identify feasible foundation types and design parameters. The preliminary subsurface information serves as the basis of the Preliminary SFER that Bridge uses to determine the most economically feasible foundation.

Once Bridge has identified the preferred bridge foundation, Geotech generates the Final SFER. The Final SFER is prepared during the Design Phase, prior to generating the final stamped bridge plans.

Note: *The preceding sequence requires Geotech to conduct the field exploration during the Preliminary Design Phase of the project. However, funding and other issues (e.g., environmental permitting) may preclude the execution of field explorations during the Preliminary Design Phase. If the field exploration is postponed until the Design Phase, the time allotted for preparing the SFER may be compressed.*

Preliminary Design Phase Interaction

Bridge and Geotech collaborate to generate the Foundation Geology Report and Preliminary SFER. Key components of this collaboration are detailed below.

Geotech Needs/Bridge Provides:

- the proposed bridge configuration (i.e., the preliminary General Layout and Site Plan drawings for the bridge options)
- the foundation locations (typically, the centerline support station and skew are shown on the Site Plan drawings)
- the total estimated factored loads (Strength I) to the foundation elements that will be used in determining reasonable sizes of foundation elements and requisite subsurface testing depths
- the total estimated Service I loads to the foundation elements

- the average unfactored pile dead load for each substructure unit, used for neutral plane method analysis
- a list of special bridge needs and concerns, if any (e.g., “limit support settlements for the proposed structure to approximately one inch under Service Load combinations” or “the existing bridge has shown signs of frost jacking at Pier 2”)
- an estimate of the scour depth at in-water piers (a method for estimating local pier scour is provided in Figure 17-1 to facilitate preliminary design in advance of a formal bridge hydraulic study); and
- historic subsurface and pile driving data (Bridge may have historic pile driving records or other relevant information in its files that may aid in the development of foundation recommendations. If such data exists, send copies to Geotech).

Bridge Needs/Geotech Provides:

- the subsurface exploration plan (the PM, responsible for controlling the project’s scope, schedule, and budget, must formally approve the plan; a copy of the plan is typically sent to Bridge for comment); and
- the Preliminary SFER, described in Section 17.2.3, containing an array of deep and shallow foundation options (feasible foundation types are examined to determine the most cost-effective structure. It is important that an ample variety of foundation recommendations be prepared to allow for meaningful cost comparisons).

Design Phase Interaction

Ideally, Bridge receives the Final SFER two months before the stamped PS&E due date. Collaboration between Bridge and Geotech is required to generate the Final SFER. Key components of this exchange are as follows:

Geotech Needs/Bridge Provides:

- the review of PS&E documents (typically distributed by PM to Geotech as part of the Review PS&E process)
- the final total factored loads (Strength I) to the foundation (these values will be provided in the foundation Data Table on the Site Plan drawing)
- the final total Service I loads to the foundation for settlement analysis, if necessary
- the final scour depth (these values will be provided in the Hydraulic and Hydrologic Summary table on the Site Plan drawing); and
- The final average pile dead load per substructure unit for neutral plane method analysis

Bridge needs/Geotech provides:

- the stamped Final SFER containing the final Foundation Geology Report as described in Section 17.2.3;
- the final pile driving special provisions, if necessary (e.g., field monitoring requirements, pile driving concerns such as hard driving, pile tip reinforcement requirements, pre-boring requirements, etc., that are not addressed in the *Alaska Standard Specifications*; and
- comments on the foundation design shown in the plans (Geotech will verify that the bridge foundation agrees with the Final SFER recommendations).

17.2.3. Content Requirements of the SFER

The Preliminary SFER and Final SFER contain the information presented in the following subsections. The Preliminary SFER focuses on design recommendations such as foundation capacity charts and feasible foundation types. The Final SFER is a fully developed report with supporting analysis and documentation.

Requirements of the Preliminary SFER

The Preliminary SFER provides geotechnical design data and recommendations for deep and/or shallow foundations.

Geotechnical Data

The Preliminary SFER contains the following geotechnical data:

- the preliminary Foundation Geology Report, including test hole locations, geological description of soils and rock, Standard Penetration Test (SPT) data, ground water table locations, temperature data, permafrost depth, and other data as applicable;

- the description of bedrock properties when present, including planes of weakness, joints, faults, rock type, Rock Quality Designation

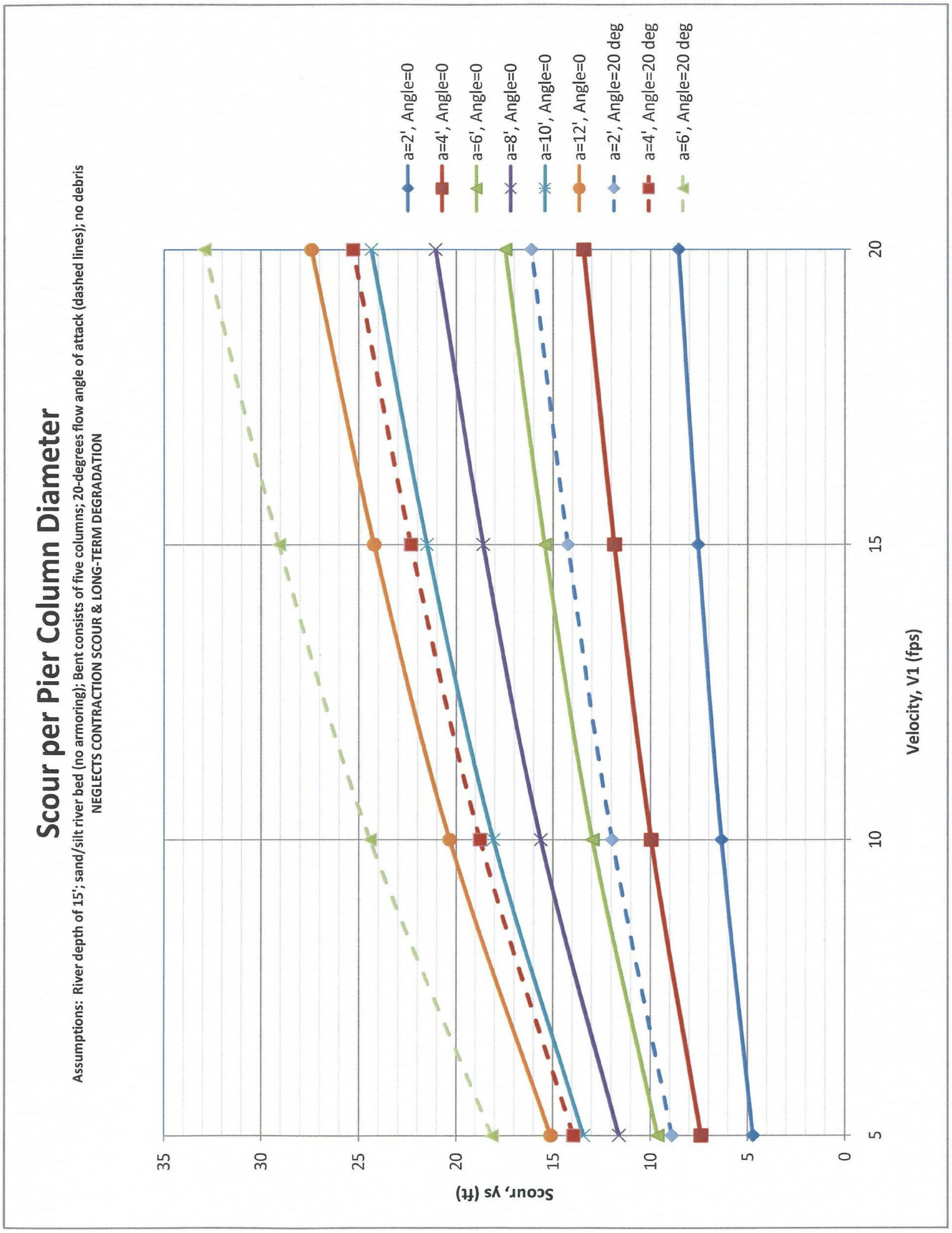


Figure 17-1
Preliminary Pier Scour Estimation Graph

(RQD), etc., as they relate to the foundation recommendations;

- the subsurface soil description, including unit weight, relative density, moisture content, phi angle, and lateral stiffness parameters and modeling recommendations for each layer of soil (Bridge will perform the lateral pile/shaft analysis);

- the presence of permafrost, high ground water table, and soil stability considerations; and

- the AASHTO seismic site class designation (i.e., "A" through "E" and, in special cases, "F") and the applicability of code-specified seismic response spectra (i.e., are there local faults that would result in seismic demands greater than those provided in the *AASHTO LRFD Bridge Design Specifications*?).

Deep Foundation Data

Typically, use deep foundations (e.g., steel H-piles, steel pipe piles, drilled shafts) at water crossings, in poor soils, and in other locations where shallow foundations are inappropriate. Preliminary design recommendations on a variety of driven pile and shaft sizes are required to determine the most cost-effective bridge foundation and bridge type. The Preliminary SFER contains deep foundation recommendations including:

- capacity tables and charts presenting the axial and uplift vertical resistance, including scour effects, as a function of embedment depth (this data is used to establish the Estimated Pile Tip Elevation for piles or the Tip Elevation for drilled shafts);

- capacity tables and charts presenting the axial and uplift vertical resistance, excluding scour effects, as a function of embedment depth (this data is used for establishing the vertical resistance at the time of construction, without regard to scour or other reductions in vertical resistance);

- capacity tables and charts presenting the axial and uplift vertical resistance, including liquefaction effects, as a function of embedment depth (the effects of scour and liquefaction may act concurrently);

- capacity tables and charts presenting the non-seismic nominal downdrag load (e.g., settlement, consolidation) either as a single value or as a function of embedment, as appropriate; and

- capacity tables and charts presenting the nominal seismic-induced downdrag load (primarily due to liquefaction effects) presented as either a single value or as a function of embedment, as appropriate.

For driven pile foundations, use the unfactored nominal resistance when preparing the vertical capacity with depth tables or charts. For drilled shafts, use the factored nominal resistance when preparing the vertical resistance with depth tables or charts. Deep foundation recommendations account for the following:

- Scour effects that reduce the amount of soil around the pile or shaft, reducing the member's vertical and lateral resistance. The Hydraulic and Hydrologic Report addresses scour effects and are summarized in the Hydraulic and Hydrologic Summary table on the bridge Site Plan drawing. For the Preliminary SFER, use the graph provided in Figure 17-1 to estimate scour effects. Figure 17-1 relates stream velocity and depth to estimated scour depth. In lieu of more accurate information, assume that the water flow velocity, V_1, is 15 feet/second. For multiple-column, pile-extension piers, assume a 20-degree water flow angle of attack (labeled "Angle = 20" on the chart). For single-column piers, assume a 0-degree water flow angle of attack (labeled "Angle = 0." The value "a" is the pile or shaft diameter). For the Final SFER, use the scour values provided in the Hydraulic and Hydrologic Summary table as the basis of the design.

- Liquefaction effects caused by seismic-induced ground motion that reduce the member's vertical and lateral resistance. The Preliminary SFER includes the soil's liquefaction potential (i.e., high, medium, low), liquefied soil properties, deformations due to lateral soil flow and settlement, and subsequent downdrag loads. (Bridge does not typically use steel H-piles or shallow foundations in liquefiable soils where lateral spread is possible).

- Downdrag loads that reduce the member's vertical and lateral resistance. Geotech shall provide recommendations for addressing downdrag (e.g., "sleeve the uppermost 10 feet of the pile" or "as required in Section 505-3.09 of the *Alaska Standard Specifications*").

- Address the spacing and group effects that would have a tendency to reduce the vertical and lateral capacity of the piles or shafts and/or minimum pile spacing.

- Rock socket length that may be required to develop vertical or lateral resistance. Provide the minimum rock socket length. (Collaboration between Bridge and Geotech may be required in establishing the rock socket length in high seismic hazard areas where the development of the member's overstrength capacity is required).

- The Preliminary SFER shall address other foundation demands such as those associated with frost jacking and heave and shall provide design recommendations.

All DOT&PF projects require field monitoring of pile driving operations. For driven pile foundations, the DOT&PF will specify the use of either:

- the "Wave equation analysis without pile dynamic measurements," or
- a "Driving criteria established by dynamic test with signal matching."

Use the corresponding dynamic analysis resistance factors, φ_{dyn}, from the most current edition of the *AASHTO LRFD Bridge Design Specifications* (currently, φ_{dyn}= 0.50 and φ_{dyn} = 0.65, for "Wave equation analysis without pile dynamic measurements" and "Driving criteria established by dynamic test with signal matching," respectively). The Preliminary SFER should include recommendations for field monitoring. In the absence of field monitoring recommendations, Bridge will determine field-monitoring requirements based on the most cost-effective option.

Shallow Foundation Data

Shallow foundations are typically used for non-water crossings (e.g., highway interchanges) where the underlying soil has good bearing capacity. The Preliminary SFER contains shallow foundation design recommendations including:

- nominal soil bearing resistance at the Service, Strength, and Extreme Event limit states as a function of effective footing width;

- minimum embedment depth required due to frost penetration and other factors affecting the nominal soil bearing resistance (in most cases, Bridge will require that the bottom of the footing be at least 3 feet below the finished ground line);

- need for replacement of the existing soil with engineered material (in some cases, the existing soil may be replaced with the structural fill material identified in the *Alaska Standard Specifications*); and

- ground water table location and its effects on the nominal soil bearing capacity (use the highest anticipated ground water table when determining the nominal bearing resistance).

Requirements of the Final SFER

The recommendations in the Final SFER are the same as those in the Preliminary SFER, except that the Final SFER addresses only the bridge foundation elements used in the final bridge design. Develop the full body of the text in the Final SFER, expounding upon:

- geotechnical data and interpretation
- discussion of foundation recommendations
- seismic conditions and liquefaction
- analysis methods and limitations
- construction issues and recommendations
- sealed and signed test hole location and boring log plan sheets
- references

Bridge cannot submit the stamped PS&E to the PM before receiving the Final SFER.

17.2.4. Plan Set Information

Provide the following information on either the bridge Site Plan drawing or, if present, the Foundation Plan drawing.

Foundation Data Tables

Bridge will provide the following table in all bridge plans using piles as a foundation element. The special provisions provide the level of field monitoring, and

Table 17-1
Pile Data Example Table

PILE DATA TABLE							
		Driving Criteria			Design Data		
Location	**Pile Type**	**Minimum Penetration (FT)**	**Estimated Pile Tip Elevation (FT)**	**Minimum Driving Resistance (K)**	**Strength I Factored Load (K)**	**Nominal Resistance (K)**	**Resistance Factor, φ**
Abut. 1	HP14 × 117	40.0	1415.0	600	350	550	0.65
Pier 2	4′-0″ × 1″ Pipe	60.0	1400.0	1400	800	1250	0.65

Table 17-2
Drilled Shaft Data Table Example

DRILLED SHAFT DATA TABLE							
		Installation Criteria			Design Data		
Location	**Shaft Diameter**	**Tip Elevation (FT)**	**Minimum Rock Socket Length (FT)**	**Minimum Top of Rock Socket Elevation (FT)**	**Strength I Factored Load (K)**	**Nominal Resistance (K)**	**Resistance Factor**
Pier 2	8′-0″	1624.0	16.0	1640.0	2100	4200	0.5

Table 17-3
Footing Pressure Table Example

FOOTING PRESSURE TABLE			
Location	**Strength I Factored Load (KSF)**	**Nominal Bearing Resistance (KSF)**	**Bearing Resistance Factor, φ**
Abut. 1	4.2	12.0	0.45
Abut. 3	4.9	12.0	0.45

provide the associated Resistance Factor in the Pile Data (see Table 17-1 for an example). Currently, a Resistance Factor value of 0.50 indicates that “Wave equation analysis without pile dynamic measurements” will be used. A Resistance Factor value of 0.65 indicates that a “Driving criteria established by dynamic test with signal matching” is required.

Minimum penetration of the pile is typically based upon lateral resistance requirements (e.g., seismic or ice demands). Base the Estimated Pile Tip Elevation upon the factored estimated resistance after scour, downdrag, liquefaction, and all other pile resistance conditions have been considered. Because scour, downdrag, and other pile resistance reductions are not present during pile driving, the Minimum Driving Resistance, in most cases, will be greater than the Nominal Resistance.

The Nominal Resistance of the pile is the anticipated pile capacity after all applicable pile resistance reductions have occurred. The Strength I Factored Load must be less than the Nominal Resistance multiplied by the Resistance Factor.

Bridge will provide a Drilled Shaft Data table (see Table 17-2 for an example) in all bridge plans that use drilled shaft foundations.

Provide the drilled shaft Tip Elevation and Minimum Rock Socket Length in the SFER. Do not include the material that is encountered above the specified Minimum Top of Rock Socket Elevation in the

Minimum Rock Socket Length (e.g., in Table 17-2, rock encountered above elevation 1640.0 does not contribute towards the 16.0-foot Minimum Rock Socket Length). If rock is not anticipated, then the table will be provided with "NA."

The Nominal Resistance of the drilled shaft is the anticipated shaft capacity after all applicable reductions have occurred. The Strength I Factored Load must be less than the Nominal Resistance multiplied by the appropriate Resistance Factor(s).

Bridge will provide the level of field inspection for drilled shafts (e.g., down-hole inspection and bottom cleanliness) in the special provisions.

Bridge will provide a Footing Pressure table (see Table 17-3 for an example) in all bridge plans that use shallow foundations.

Seismic Parameters

Bridge will provide the seismic design parameters, as shown in Figure 17-2, in the "GENERAL NOTES" of the Site Plan drawing. Provide the spectral acceleration values in the *AASHTO LRFD Bridge Design Specifications* and the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Provide the Site Class and Liquefaction Potential in the SFER

For Site Class F soils or other situations where a site-specific response spectra is used in the bridge design, include the site-specific spectra on the bridge plans sheets.

Log of Test Hole Borings

Bridge will incorporate the sealed and signed test hole location and boring log plan sheets in the final bridge plans.

17.2.5. Construction Support

Both Geotech and Bridge must be available during construction to address construction-related foundation and geotechnical questions and problems and to provide technical advice to the Construction Project Engineer.

For pile foundations, Geotech will be required to:

- review the adequacy of the Contractor's proposed pile driving plan;
- review the adequacy of the Contractor's proposed pile driving hammer;
- provide the pile driving acceptance criteria (also known as the inspector's chart) when a "*Wave equation analysis without pile dynamic measurements*" is specified;
- provide preliminary pile driving acceptance criteria when *"Driving criteria established by dynamic test with signal matching"* is specified; and
- generate the scope of services for the Construction Project Engineer when a *"Driving criteria established by dynamic test with signal matching"* is specified and interact with the PDA consultant once its services have been acquired.

For drilled shaft foundations, both Geotech and Bridge will be required to:

- review the adequacy of the Contractor's proposed shaft installation plan; and
- review field inspection reports (e.g., shaft cleanliness, cross-hole-sonic logs).

For shallow foundations, Geotech may be required to evaluate foundation adequacy when the actual soils deviate from those presented in the Foundation Geology Report and Final SFER (e.g., groundwater table, rock characteristics, soil type).

SEISMIC PARAMETERS

PGA = 0.25
$S_s = 0.65$
$S_1 = 0.20$
Site Class = C
Liquefaction Potential = Low
AASHTO 7% probability of exceedence in 75 years

Figure 17-2
Seismic Parameters

17.3. Footings and Caps

17.3.1. Terms

Spread Footing: A slab of concrete directly transferring load to the soil beneath it.

Pile Caps: A strip of concrete transferring load to a single row of piles.

Pile Footings: A slab of concrete transferring load to multiple rows of piles.

17.3.2. General

The following criteria apply to both footings and caps.

Basic Design Criteria
Reference: LRFD Articles 5.12.8.6 and 5.12.8.7.

The footing or cap thickness may be governed by the development length of the column or wall reinforcement, or by seismic requirements.

Construction Joints
Footings and caps do not generally require construction joints. Where used, offset construction joints 2 feet from expansion joints or construction joints in walls, and construct the joints with keyways.

Stepped Footings/Caps
Stepped footings and caps are only used occasionally. Where used, the difference in elevation of adjacent stepped footings or caps should not be less than 4 feet. See Figure 17-3.

Depth
Locate pile caps or footings above the lowest anticipated scour level if the piles are designed for this condition. Construct caps or footings to neither pose an obstacle to water traffic nor be exposed to view during low flow. Construct caps or footings to pose a minimum obstruction to water and debris flow if exposed during high flows.

17.3.3. Spread Footings

Embed spread footings a sufficient depth to provide the greatest of the following:

- adequate bearing, scour, and frost heave protection (typically defined in the SFER);
- 3 feet to the bottom of the footing; or
- 2 feet of cover over the footing.

Sliding Resistance
Reference: LRFD Article 10.6.3.4.

Except for unusual cases, do not use keys in footings to develop passive pressure against sliding. When it becomes necessary to use a key, the bridge engineer should consult with the Statewide Materials Section.

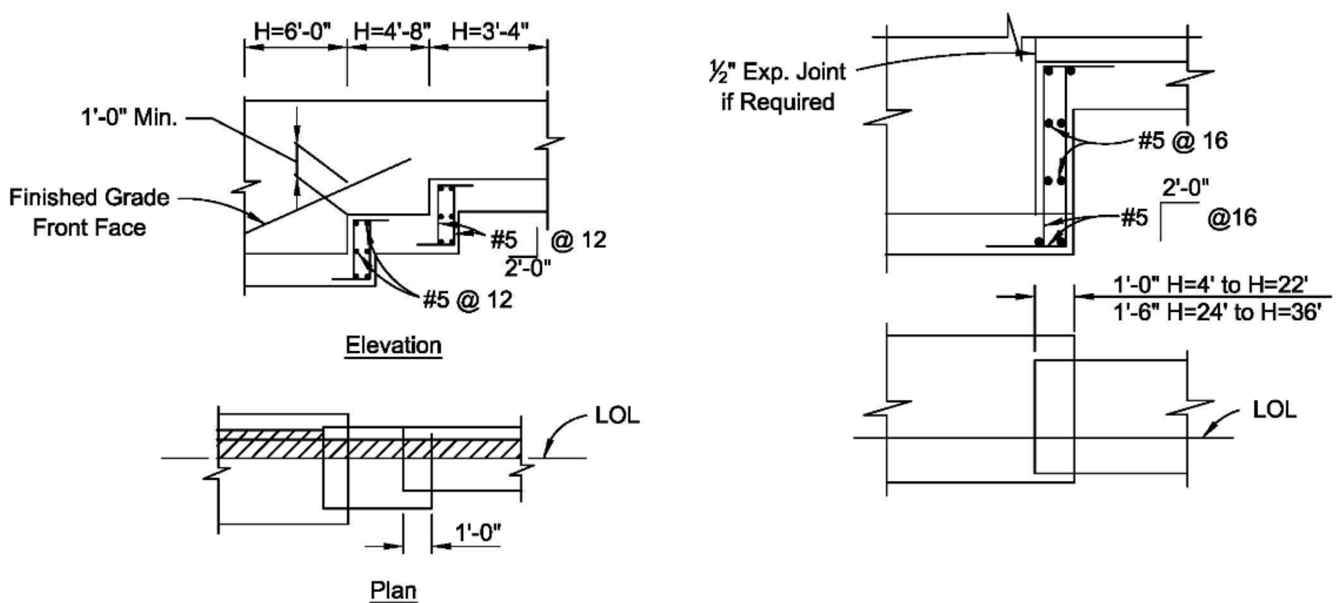


Figure 17-3
Stepped Footings/Caps Pile Caps/Footings

17.4. Driven Piles

Piles serve to transfer loads to deeper suitable strata. Piles may function through skin friction, through end bearing, or a combination of both.

17.4.1. Pile Types/Selection

See Section 11.7.4 for DOT&PF practices for selecting driven piles as the foundation type. Minimize the use of differing pile types and sizes.

Steel H-Piles

DOT&PF uses steel H-piles to support abutments protected against scour, where a competent bearing layer is available. The steel H-pile shape most commonly used by DOT&PF is HP14x117. Other sizes may be acceptable. Where a significant savings may be realized by using non-typical sizes or where the design dictates, use other standard AISC sizes.

Steel Pipe Piles

Reference: LRFD Articles 6.9.5 and 6.12.2.3.

DOT&PF uses steel pipe piles in waterways where the predicted scour is deep and driving conditions are favorable, and at sites prone to liquefaction. Use pipe pile extension piers to speed construction compared to the use of pile footings with CIP concrete columns. Conventionally, use open-ended piles where cobbles and boulders may be encountered during driving. This allows use of a down hole hammer to break up obstructions.

Use the following specifications for structural steel pipe piles:

- DOT&PF Special Provisions for spiral welded pipe piles.
- American Petroleum Institute (API) Specification 5L X52 PSL2, Specification for Line Pipe.
- API 2B using ASTM A709 Grade 50T3.
- ASTM A53 Grade B.

Serious shortcomings in the traditionally applied ASTM A252 standard have been identified when attempting to specify steel pipe piles adequate for current bridge design and construction practices.

The following also applies:

1. **Diameter**. DOT&PF uses pipe pile diameters of 12 inches to 48 inches. The wall thickness typically is not less than 1:48 of the pipe diameter.
2. **Interior Filler**. Typically, fill steel pipe piles with reinforced concrete to strengthen and stiffen the pipe and as a means of connecting the pile to the cap/footing.

Pile Selection

The Bridge Section selects the pile type based on the SFER. Figure 17-4 provides guidance in selecting pile types based on their typical usage by DOT&PF.

17.4.2. Design Details

Reference: LRFD Article 10.7.1.

Pile Length

Determine pile length based on the SFER. All piles for a specific pier or abutment should be the same length where practical. Show pile lengths in whole-foot increments.

The estimated pile tip elevations and minimum penetration will be shown on the Pile Data Table in the contract documents. Ensure that the estimated pile tip elevations reflect the elevation where the required ultimate pile capacity is anticipated to be obtained. The minimum penetration should reflect the penetration required, considering scour and liquefaction, to support both axial and lateral loads.

Piles placed at abutment embankments that are more than 5 feet in depth require pre-drilling. The size of the pre-drilled hole is 2 inches larger than the diameter or largest dimension of the pile.

Reinforced Pile Tips

Use reinforced pile tips to minimize pile damage where hard layers are anticipated and as recommended in the SFER. Where rock is anticipated, designate that the pile tips will be equipped with teeth designed to penetrate into the rock. Show the type of pile tip reinforcing on the plans.

Battered Piles

Do not use battered piles due to their past poor performance in moderate to high seismic areas.

Pile Footing and Cap Details

The following applies to the connection of piles to pile caps or to pier caps unless seismic analysis dictates otherwise:

1. **Steel H-Piles**. See Figure 17-5.

2. **Steel Pipe Piles**. Always extend longitudinal column bars to the top of the cap. Fully develop the reinforcing steel through adequate development length or standard hooks. See Figure 17-6.

17.4.3. Force Effects

Uplift Forces

Lateral loads can cause uplift forces (e.g., seismic forces, buoyancy, frost jacking). Check piles intended to resist uplift forces for resistance to pullout and for structural resistance to tensile loads. Check the connection of the pile to the cap or footing.

Laterally Loaded Piles

Section 17.6 discusses pile analysis for lateral loading and resistance.

17.4.4. Pile Loads

Show applicable pile loads on the plans. This information will help ensure that pile driving efforts will result in a foundation adequate to support the design loads; see Section 17.2.4 for Foundation Data Table requirements.

Pile Type	Soil Conditions and Structural Requirements
Steel H-pile	Rock or dense soil where end bearing is desirable and lateral flexibility in one direction is not critical. Common at abutments and for pile footings, but not typically used in liquefiable soils.
Steel pipe pile (closed or open end)	Loose to medium dense soils or clays where skin friction is the primary resistance and lateral stiffness in both directions is desirable, especially in rivers where deep scour or liquefaction is anticipated and high lateral stiffness is needed.

Figure 17-4
Driven Pile Selection

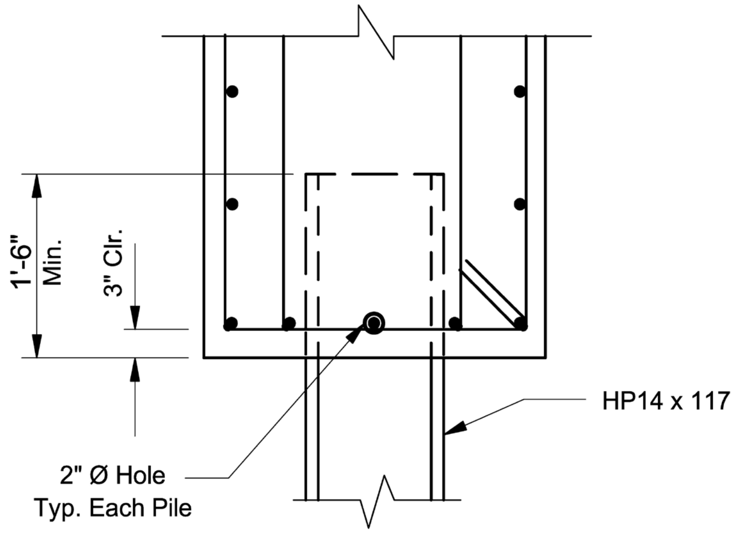


Figure 17-5
Steel H-Pile Connection

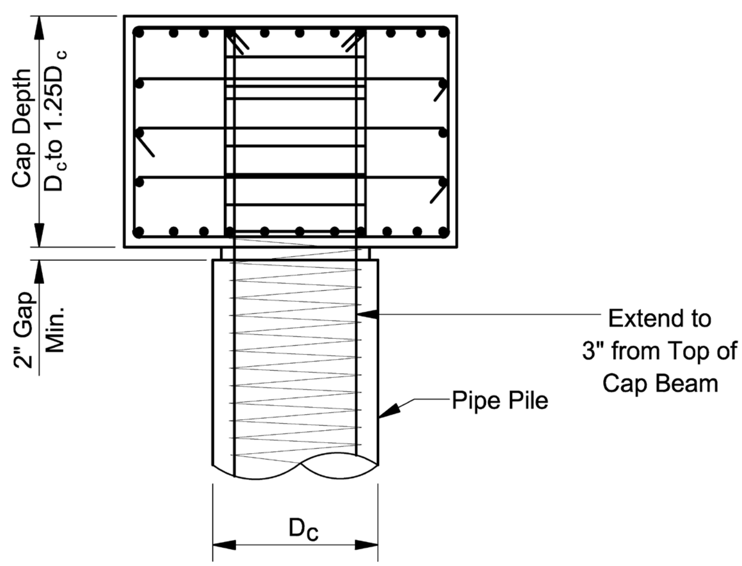


Figure 17-6
Steel Pipe Pile Connection

17.5. Drilled Shafts

17.5.1. Usage

Guidance for selecting drilled shafts as the foundation type can be found in Chapter 11 of this Manual.

Drilled shafts derive load resistance either as end-bearing shafts transferring load by tip resistance or as friction shafts transferring load by side resistance or a combination of both.

17.5.2. Drilled Shaft Axial Compressive Resistance at the Strength Limit State

The *LRFD Specifications* provide procedures to estimate the axial resistance of drilled shafts in cohesive soils and cohesionless soils in Articles 10.8.3.5.1 and 10.8.3.5.2. In both cases, the resistance is the sum of the shaft and tip resistances. LRFD Article 10.8.3.5.4 discusses the determination of axial resistance of drilled shafts in rock.

17.5.3. Structural Design

Column Design. Because even soft soils provide sufficient support to prevent lateral buckling of the shaft, design drilled shafts surrounded by soil according to the criteria for short columns in LRFD Article 5.6.4.4 when soil liquefaction is not anticipated. If the drilled shaft is extended above ground to form a pier, design the shaft as a column. Similarly, consider the effects of scour around the shafts in the analysis.

Casing. DOT&PF almost always uses a permanent casing to maintain the excavation, especially when placing a shaft within the groundwater table. Use the casing in the determination of the structural resistance of the shaft, depending on the thickness of the casing. In seismic analysis and design, use a strain compatibility method to determine the stiffness and strength of the cased shaft.

Lateral Loading. Section 17.6 discusses drilled shaft analysis for lateral loading and resistance.

17.5.4. Design Details

1. **Diameter**. The diameter of a drilled shaft supporting a single column should be at least 18 inches greater than the greatest dimension of the column cross section. Shaft diameters up to 120 inches have been used in Alaska.

2. **Location of Top of Shaft**. Typically, terminate drilled shafts 6 inches above the finished grade or at 12 inches above the water elevation anticipated during construction.

3. **Reinforcement**. Section 14.2 discusses DOT&PF practices for the reinforcement of structural concrete, which apply to the design of drilled shafts. Additional reinforcement criteria include:

   a. For the shaft, provide a minimum reinforcement of 1 percent of the gross concrete area. Extend the shaft reinforcement from the bottom of the shaft into the footing, if present.

   b. For confinement reinforcement, use spirals (up to #7) or butt-welded hoops.

   c. The design and detailing of drilled shafts must provide clearances of 4 inches to 6 inches for reinforced steel cages. Maintain the annular space around the cage with non-corrosive spacers.

   d. Detail drilled shafts and columns to accommodate concrete placement considering the multiple layers of reinforcing steel including lap splices. Maximize lateral reinforcement spacing.

   e. Strive to provide windows about 5-inch square between longitudinal and transverse bars.

   Figures 17-7 and 17-8 illustrate the typical drilled shaft and column longitudinal and transverse reinforcement.

4. **Class DS Concrete**. Construct all drilled shafts with Class DS Concrete, Concrete for Drilled Shaft Foundations. See Section 501 of the *Alaska Standard Specifications for Highway Construction* for these concrete material requirements. Class DS Concrete includes smaller aggregates and provides greater slump, among other features, to facilitate placement of the concrete into the drilled shaft.

5. **Construction Joints**. Do not use construction joints for drilled shafts except with approval by the Chief Bridge Engineer.

6. **Casing**. DOT&PF typically uses permanent metal casings for drilled shafts.

7. **Constructability**. Detail drilled shafts and columns to accommodate concrete placement through the layers of reinforcing steel. Limit lap splices in the drilled shaft locations and provide adequate openings. The objective is to provide windows between horizontal and vertical bars equal to 5-inch squares.

8. **Rock-Socketed Shafts**. Where casing through overburden soils is required, design the shaft as one size and, if necessary, step down (reduce the diameter) when going into a rock socket.

9. **Specialized Contractors**. Use specialized contractors for drilled shafts greater than 6 feet in diameter.

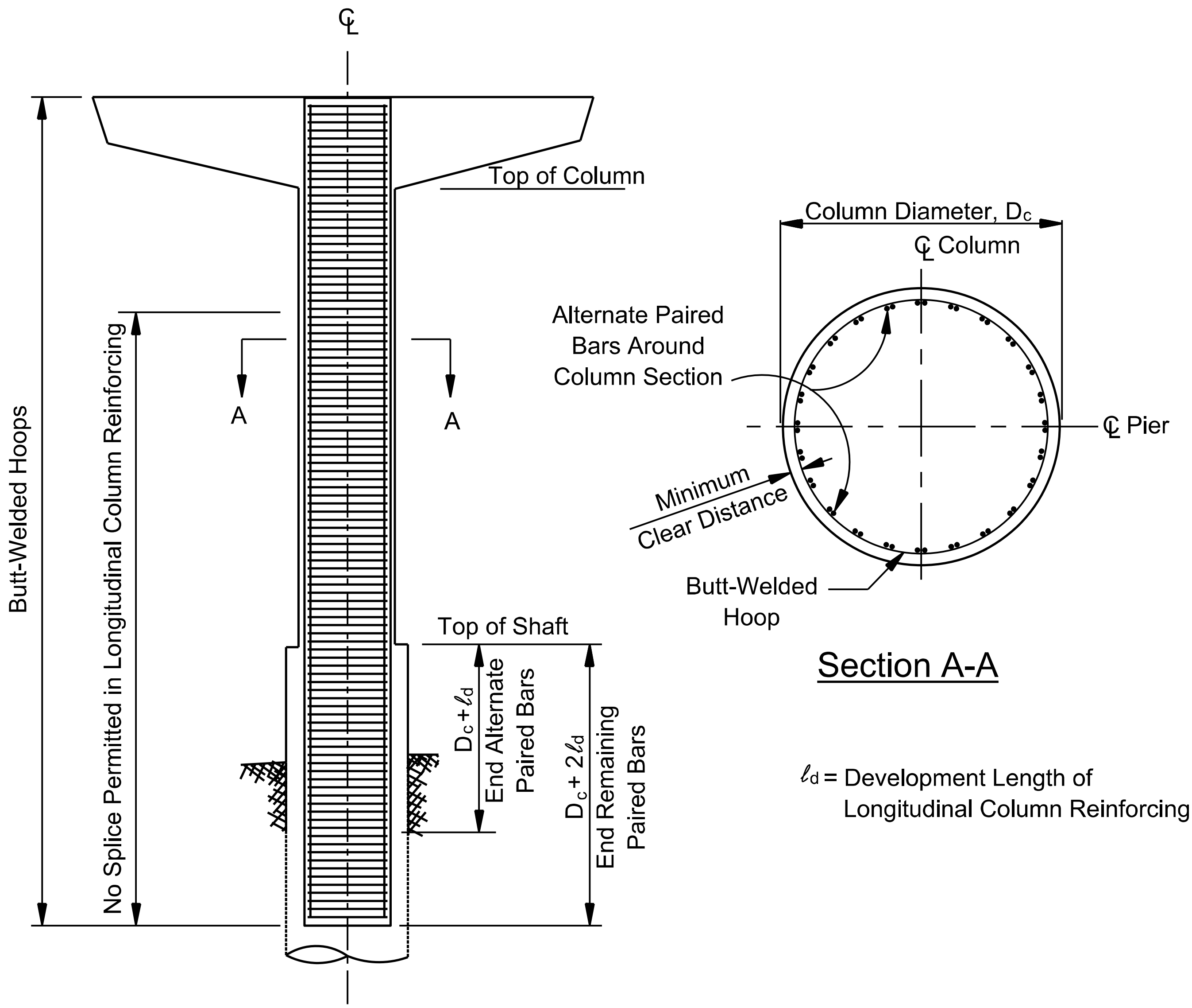


Figure 17-7
Typical Drilled Shaft

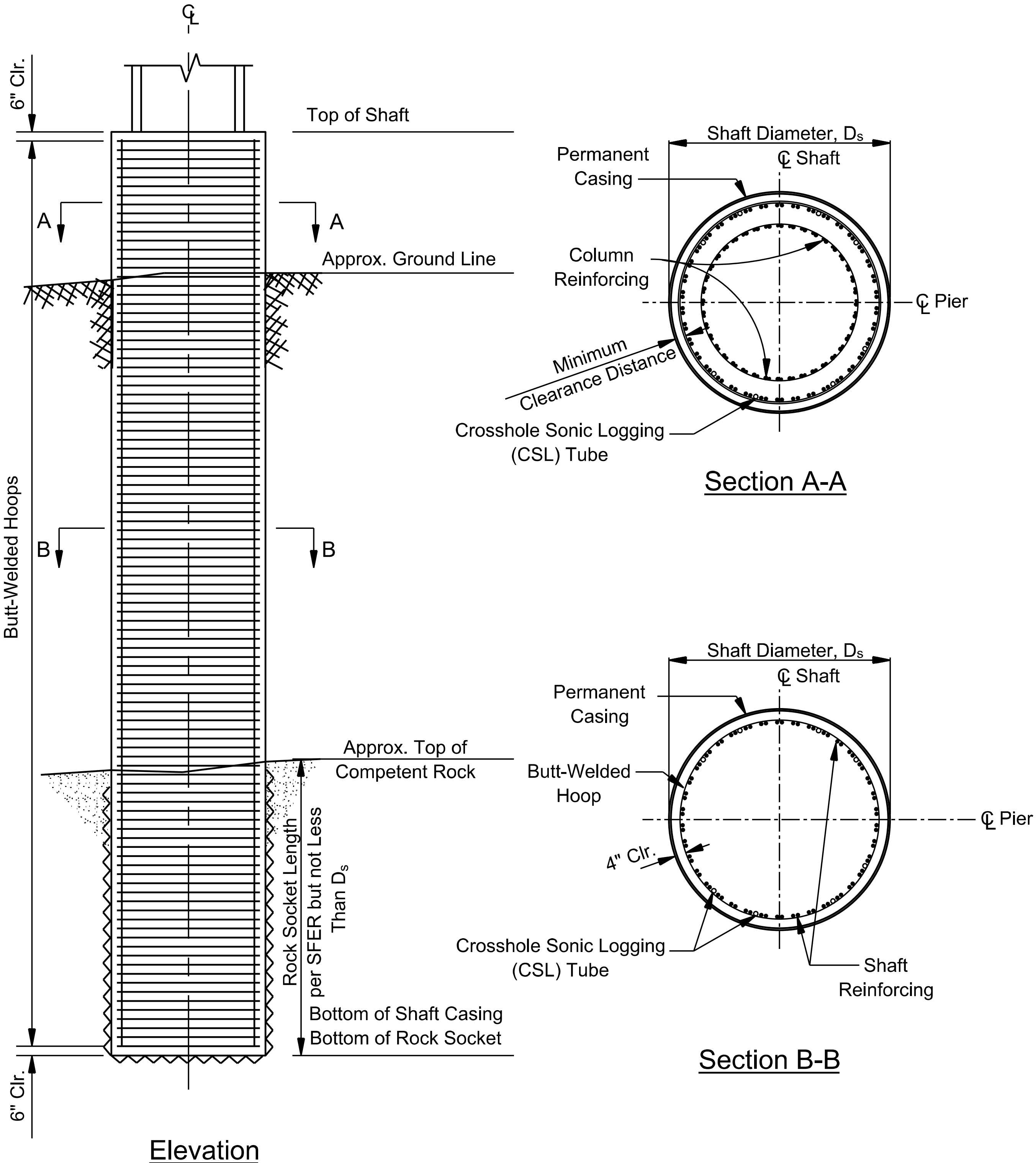


Figure 17-8
Drilled Shaft
(Socketed in Rock)

17.6. Lateral Loading of Deep Foundation Elements

17.6.1. *Pile/Shaft Supported Footings*

Pile and shaft supported footings typically behave as fixed supports, and the lateral stiffness of deep foundation elements does not typically need to be considered in the non-seismic design of these elements. Lateral stiffness of the deep foundation elements may need to be included in the seismic analysis of the bridge when the bridge engineer anticipates soft soils, liquefaction, or other factors that affect the lateral stiffness of the footing. Use the modeling techniques presented in Section 17.6.2 to determine the lateral stiffness of deep foundation elements.

17.6.2. *Pile/Shaft Extension Piers*

Include the lateral stiffness of deep foundations in both the non-seismic (small lateral deflection) and seismic (large lateral deflection) of pile/shaft extension piers. Include the effects of scour, liquefaction, and frozen soil, when applicable, in the lateral stiffness analysis.

Several methods of analysis are available for calculating the lateral stiffness of deep foundation elements. Not all of the methods discussed below are applicable to all situations, and the bridge engineer should be aware of each method's limitations.

Closed-Form Linear Models

For small lateral deflections, closed-form solutions have been developed based upon a "beam on an elastic foundation" model. These methods provide a depth to effective fixity for moment (l_m) and deflection (l_s) wherein the actual soil-pile system is replaced by an equivalent fixed-base cantilever. LRFD Article C10.7.3.13.4 and Figure 17-9 provide the equations describing these systems for both cohesive and cohesionless soils. These equations are often referred to as the "fourth-root" or "fifth-root" equation, depending upon the soil type. These equations typically provide sufficiently accurate results for most situations where the deflections are small and the response is elastic.

Closed-form solutions also exist for large-deflection stiffness determination but, like most hand methods, are not readily capable of addressing soil layering and other "real-world" variability. Nonetheless, these methods provide a good means of checking the more sophisticated, computer-generated results.

Non-Linear Models

As the lateral demands increase, the soil and pile/shaft may behave in a non-linear manner. In these situations, numerical modeling of the soil-pile/soil-shaft interaction is often required. These numerical approaches are capable of incorporating the non-linear soil and structure response, but they rely upon computer software. The most commonly used software programs are FB-Pier, the DOT&PF "Pushover Program," and L-Pile. Use the results of these methods to provide a depth to effective fixity, such as that described in Section 17.6.2, or to develop an equivalent soil spring model, such as that shown in Figure 17-9.

The use of non-linear models is often required for seismic analysis.

17.6.3. *Minimum Penetration*

The Estimated Pile Tip Elevation is determined from the maximum vertical pile/shaft demands and the expected vertical pile capacity presented in the SFER. Verify the vertical pile capacity in the field by using either wave equation analysis or dynamic pile monitoring (PDA/CAPWAP).

Embed piles and shafts into the soil so that the deflected shape of the pile subjected to lateral loads crosses a zero deflection point at two places. This point may be determined by using three times the depth to effective fixity (l_s) as calculated in Section 17.6.2, or as numerically determined in the non-linear approaches described in Section 17.6.2.

In addition to crossing the zero deflection point two times, for bridges in seismic design category (SDC) B, C, and D, embed the pile sufficiently to develop the overstrength plastic hinging moment (M_{po}) of the pile or shaft unless it is otherwise capacity protected from developing below-ground hinges.

17.6.4. *Effects of Frozen Soil on Structural Response*

The upper layers of soils with high moisture content and those below the ground water table are subject to seasonal freezing. The depth of seasonally frozen ground varies around the state but is typically between two feet and ten feet. Frozen soil may be up to several orders of magnitude stiffer and stronger than unfrozen soil. Include frozen soil effects in the foundation analysis when all the following conditions apply:

- The applied loads are dynamic, including seismic, vehicle collision or other Extreme Event load combinations
- Locations where temperatures at the site could remain below 32 °F continuously for more than two weeks
- Sites where the ground water table may be present within 10 feet of the ground surface.

The structural engineer should be aware that variations in the applicability of frozen soil effects are possible between different piles or substructure units of the bridge (i.e., not all substructure units may be frozen at one time). Bents with short pile extensions are particularly susceptible to frozen soil effects and should be designed to meet the requirements of Seismic Design Category D in both the unfrozen and frozen soil conditions.

The effects of frozen soil have been found to have a negligible effect on the seismic response spectra (Yang et al 2010). Do not consider the increased frozen soil stiffness when determining the site classification.

Include frozen soil stiffness in the lateral analysis of foundations and substructures. Frozen soil stiffness such as p-y curves can be provided by the foundation engineer. When subjected to quasi-static loads, such as thermal expansion or contraction, the effects of frozen soil are often negligible due to the high creep associated with frozen soils. When subjected to rapidly applied loads such as earthquake or collision, the effects of frozen soil may significantly stiffen the bridge response.

For seismic considerations and pushover analysis of pile extension bents in frozen soil, a depth to effective fixity approach (see LRFD Article 17.6) may be used (Yang et al 2012). The effective fixity for moment (L_m) and deflection (L_s) may be taken as:

$$L_m = 0.25\ D_c$$

$$L_s = 4\ D_c$$

where:

D_c = pile or shaft diameter

In lieu of a more detailed pushover analysis, the following approximate force-displacement relationships for a single free-head pile or shaft may be used:

$$P_{yi} = \frac{M_{yi}}{H + L_m}$$

$$\Delta_{yi} = \frac{\varphi_{yi}\,(H + L_s)^2}{3}$$

$$P_u = \frac{M_u}{H + L_m}$$

$$\Delta_u = \Delta_y\left(\frac{M_u}{M_{yi}}\right) + L_p(\varphi_u - \varphi_{yi})\,(H + L_m)$$

where:

P_{yi} = idealized lateral yield force

M_{yi} = idealized yield moment

H = pile or shaft height measured from the ground line to the top of the section

Δ_{yi} = idealized lateral yield displacement

Δ_y = idealized yield displacement

ϕ_{yi} = idealized yield curvature

P_u = maximum lateral force

M_u = ultimate moment

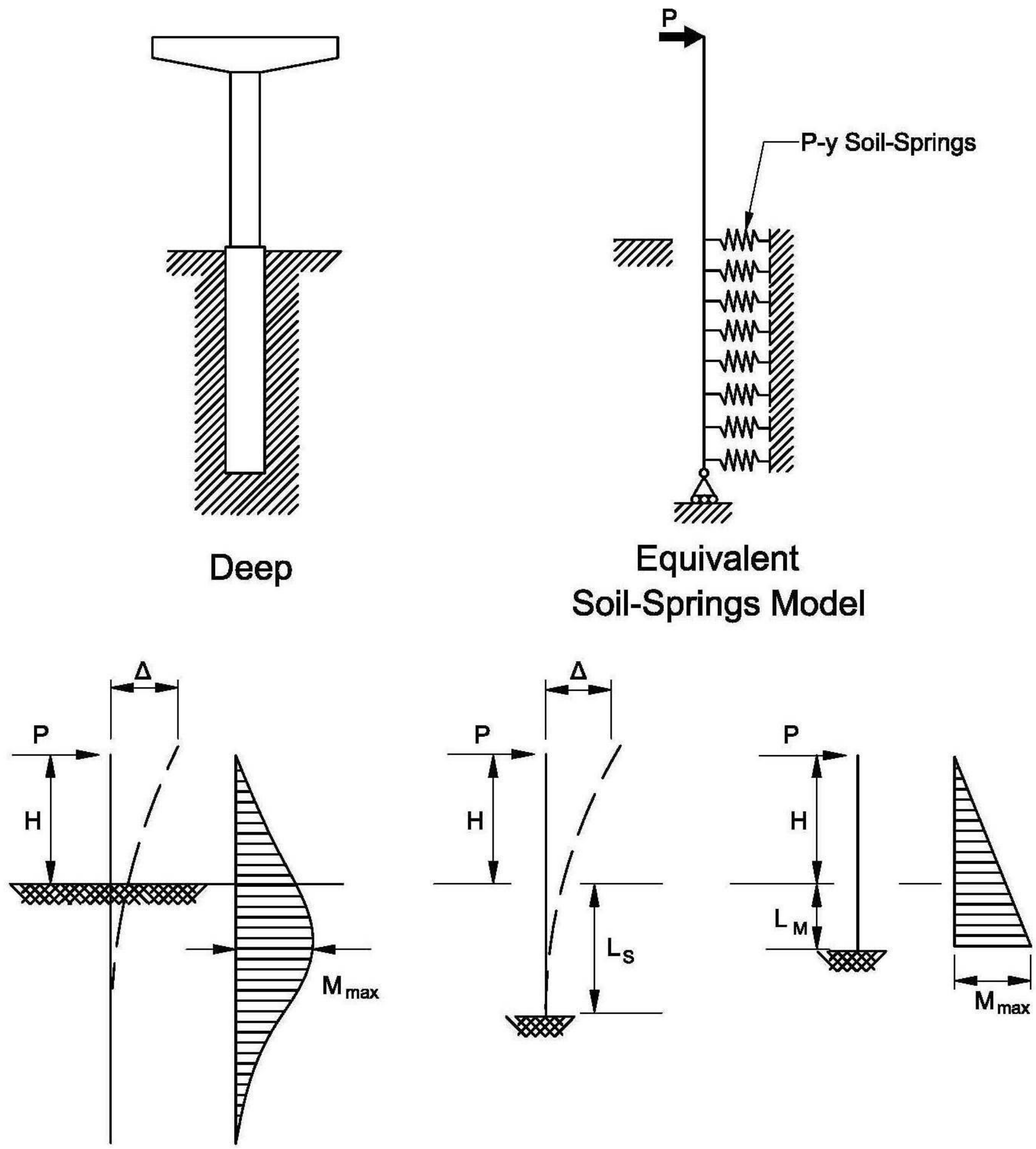
Δ_u = maximum lateral displacement capacity

L_p = analytical plastic hinge length = 2 D_c

ϕ_u = ultimate curvature

Similar equations can be developed for fixed-head piles and shafts recognizing that the height, H, can be replaced by the distance from the plastic hinge to the point of contraflexure.

Refer to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* for the generation of the moment-curvature relationship for reinforced concrete and concrete filled steel pipe sections



Coefficient n_h (Kips per cubic feet)			
Relative Density	**Loose**	**Medium**	**Dense**
Above Ground Water	14	42	112
Below Ground Water	8	28	68

	L_S	L_M
Cohesive Soil Constant k_h	$1.4\sqrt[4]{\frac{EI}{k_h}}$	$0.44\sqrt[4]{\frac{EI}{k_h}}$
Cohesionless Soil Constant n_h	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

k_h = Coefficient of horizontal subgrade reaction for fine-grained soil

$$= \frac{160\overline{m}c}{b}\left(\text{in } k/ft^2\right)$$

In which $\overline{m}$ = 0.32 for $c < 1$ ksf
= 0.36 for $1 < c < 4$ ksf
= 0.40 for $c > 4$ ksf
b = width of pile (ft)
c = soil cohesion

Figure 17-9
Method of Modeling Deep Foundation Stiffness

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18. Substructures

This chapter presents DOT&PF design information on substructure elements, which supplement the *AASHTO LRFD Specifications*. Section 11.6 of the *Alaska Bridges and Structures Manual* presents DOT&PF criteria for the selection of substructure components within the context of structure-type selection.

18.1. Abutments/Wingwalls

Various types of abutments are available to support the bridge superstructure. An abutment may include an end diaphragm, a stem wall, pile cap beam, backwall, and wingwalls.

In addition to vertical support, abutments provide lateral support for fill material on which the roadway rests immediately adjacent to the bridge.

Abutments are generally cast-in-place, reinforced concrete and founded on spread footings, drilled shafts, or driven pile footings, as appropriate for the site.

18.1.1. General Abutment Design and Detailing Criteria

The following applies to the design and detailing of abutments:

1. **Minimum Thickness**. The minimum allowable wall thickness is 12 inches.
2. **Abutment Slope**. The preferred abutment slope is 2H:1V measured normal to the centerline of bearing. This slope may sometimes be steepened to a minimum of 1½H:1V to avoid the need for a deeper prestressed concrete girder.
3. **Terminology**. An “end diaphragm” is always integral with the superstructure. The term “backwall” only applies where the wall is part of a seat abutment and, therefore, not integral with the superstructure.

18.1.2. Semi-Integral Abutments

The semi-integral abutment is DOT&PF’s preferred abutment configuration. Figures 18-1 and 18-2 present typical designs — one founded on piles and the other on a spread footing. For this type of abutment, the integral end diaphragm is cast around the girder ends and attached to the slab, but separated from the cap.

Thermal movements and live load rotations are accommodated through the bearings, girders, end diaphragms, and approach slabs.

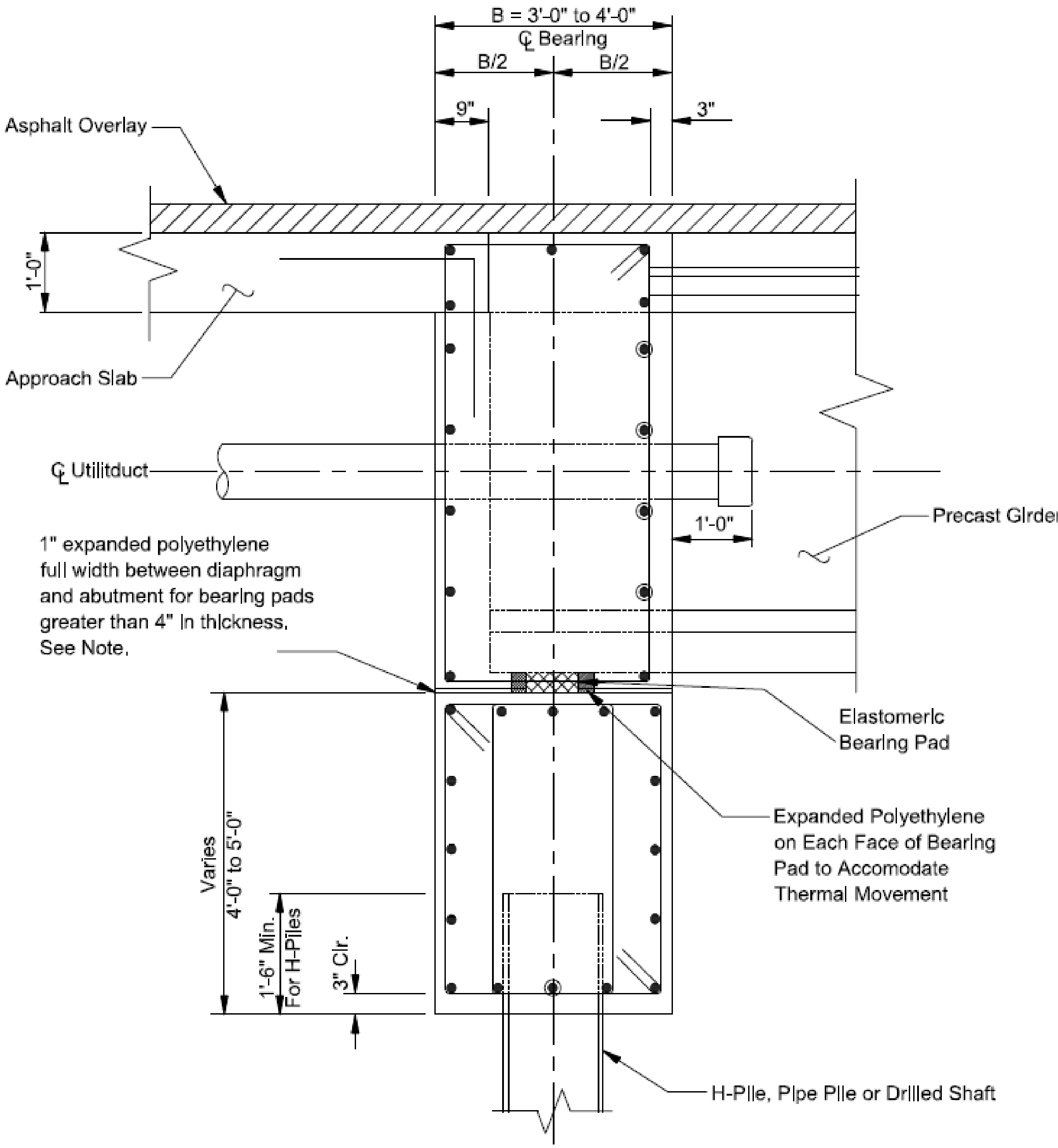
Semi-integral abutments allow diaphragm movement and rotation through the detailing of the bearing or connection of the girder, diaphragm, and the cap as either:

- **Fixed**. A pinned connection (free to rotate, fixed against translation).
- **Free**. A roller connection (free to rotate, free to translate).

Usually in a single-span bridge, the design fixes one end (typically, the downhill end) while the other end is free to translate. In a multi-span bridge, both abutments will usually be free with fixity provided at the pier(s).

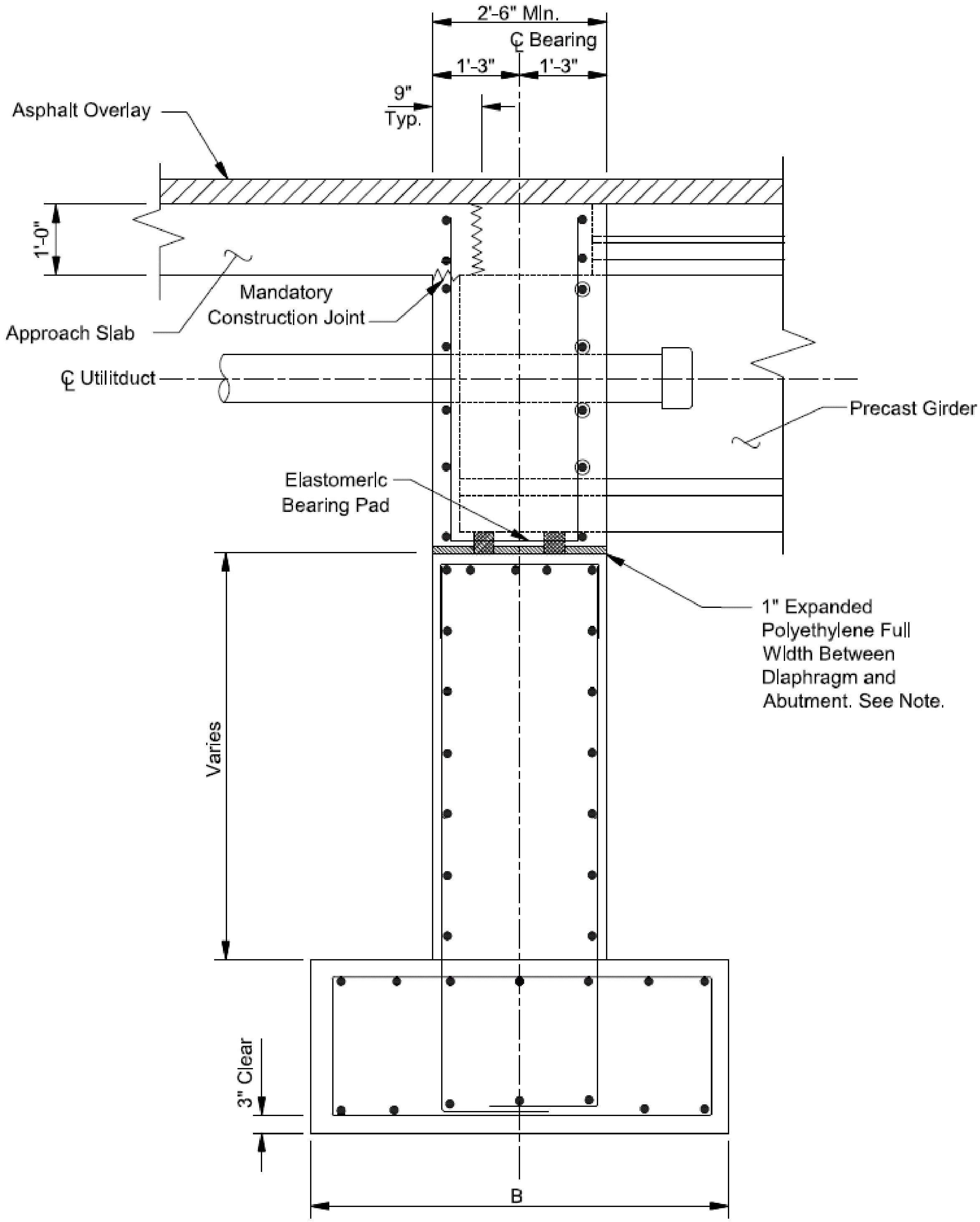
In addition, the following applies to the design of semi-integral abutments:

1. **Fixed End**. Assume a pinned (free to rotate, fixed against translation) end for the structural design of the superstructure.
2. **Diaphragm Width**. Typically, the end diaphragm width is the same as the pile cap beam but shall be a minimum of 30 inches. Commonly used pile sizes typically result in either 36-inch or 48-inch wide diaphragms for semi-integral abutments.



Note: For bearing pads less than 3" in thickness, the expanded polyethylene is the same thickness as the bearing pad.

Figure 18-1
Typical Semi-Integral Abutment
(On Piles)



Note: For bearing pads less than 3" in thickness, the expanded polyethylene is the same thickness as the bearing pad.

Figure 18-2
Typical Semi-Integral Abutment
(On Spread Footing)

18.1.3. Seat Abutments

Figure 18-3 presents a typical seat abutment. Seat width will generally be controlled by seismic design requirements, but in no case shall the seat width be less than 30 inches.

18.1.4. MSE-Wall Abutments

DOT&PF uses two basic types of abutments with MSE walls:

- **"True Abutment"**. An abutment supported by an MSE wall, in which the wall rests on a spread footing atop the reinforced earth. In the design of the MSE wall, consider the load from the spread footing as an earth surcharge load (ES). True abutments are generally limited to single-span bridges.
- **"False Abutment"**. A pile-supported abutment with the piles passing through the MSE embankment. Isolate the piles from the MSE backfill through sleeves to eliminate downdrag, and found the piles in the soils below the MSE wall. False abutments are typically used for multi-span bridges with a continuous superstructure.

Piles Within a False Abutment

Piles placed within the MSE backfill require special consideration. Ensure that the piles are placed prior to the construction of the wall.

As the wall is constructed, the subsoils beneath the wall and the MSE wall itself may compress. The piles, however, are rigid. The compression of the soils will induce a load into the piles due to friction. Depending on site materials, these downdrag forces can be substantial.

To reduce the friction on the piles and to mitigate the downdrag forces, place the piles in pile sleeves, or place a slightly larger corrugated pipe over the pile prior to backfilling. Fill the space between the pile and the corrugated pipe with pea gravel or similar free-draining material.

Modify the soil reinforcement when piles are located within the wall. Do not bend the soil reinforcement around the piles; the soil reinforcement must remain linear to develop its strength. Also, do not attach the soil reinforcement to the piles. Allow a reinforcement skew of up to 15 degrees from a line perpendicular to the wall face if the design accounts for this.

Reinforcing mats can be cut and skewed, but they must conform to the following:

- Do not allow single longitudinal wires.
- Reinforcing mats develop their strength from the cross wires. Provide at least two longitudinal wires to make the cross wire effective.
- Cut segments must meet minimum pull-out capacity factors of safety. Testing of cut segments is required to show that their full strength is developed.

The contractor must perform all cutting of reinforcement prior to the application of corrosion protection.

If cutting and skewing cannot resolve all conflicts, the bridge engineer may need to provide steel frames around the piles connecting straight soil reinforcing on either side of them.

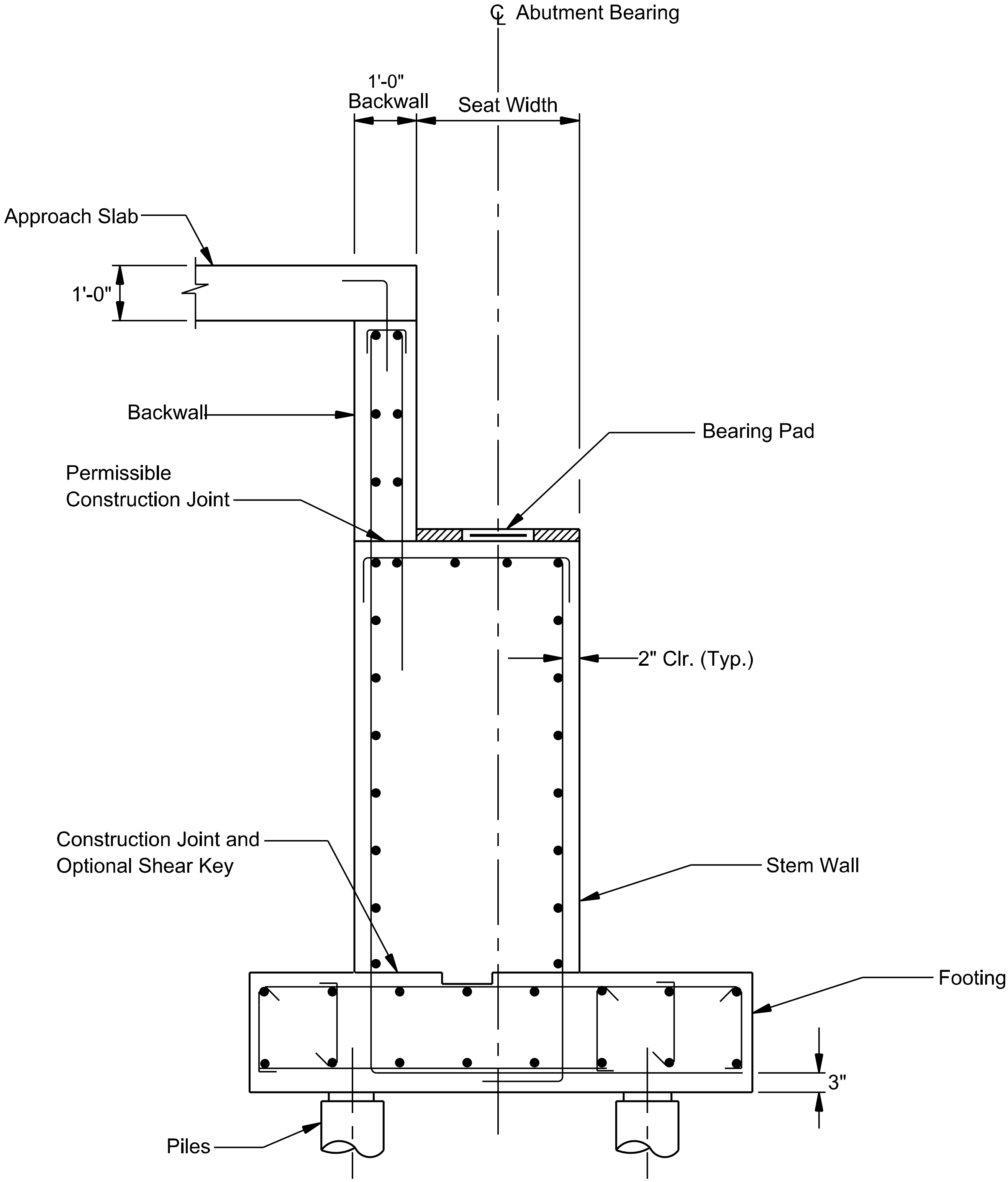
Design these frames to transfer all forces within the soil reinforcement, which must be corrosion protected. The wall supplier must detail all bridging frames in the shop drawings.

Chapter 21 discusses the use and design of MSE walls in more detail.

18.1.5. Integral Abutments

Reference: LRFD Article 11.6.2.1.

As discussed in Section 11.6.2, the Chief Bridge Engineer must approve the use of integral abutments because of their inherent incompatibility with frozen ground and extreme thermal ranges.



Note: This typical seat abutment shows piles.
Drilled shafts, H-Piles and spread footings are also options.

Figure 18-3
Typical Seat Abutment

18.1.6. Piles at Abutments

Reference: LRFD Article 10.7.

This discussion specifically addresses the use of driven piles with abutments. See Section 17.4 for additional information on piles. The following criteria apply to piles for abutments:

Number

Use the most cost-effective pile type and size, but do not use fewer than three piles to support an abutment.

Pile Type

Typically, use HP14 × 117 in soils that are not susceptible to liquefaction. Use 18-inch or larger diameter, concrete-filled pipe piles or drilled shafts in soils susceptible to liquefaction.

Pile Spacing

Pile spacing should generally not result in more than one pile per girder; however, placing a pile beneath each girder is not critical. Space the piles across the length of the abutment to help distribute abutment loads uniformly to each pile.

Cap Overhang

The minimum cap overhang is 18 inches measured from the centerline of the pile, but in all cases, the cap overhang reinforcement must be adequately developed. Hooked or headed bars may be required to develop the cap reinforcement.

Pile Loads/Forces

For the semi-integral abutment, design the end diaphragm to resist the force from the bearings and lateral earth pressure, including seismic-induced earth pressures.

Pile-Cap Connections

To allow for constructability, the pile top lateral position must have a tolerance of ±6 inches. Extend steel H-piles a minimum of 12 inches (preferably 18 inches) into the cap.

Abutments supported on pipe piles should be designed and detailed similar to pier cap beams supported on pipe pile extensions *except* that the cap beam depth may be taken greater than 125 percent of the pile diameter.

Construction

Consider the placement tolerances for all abutment types and ensure pile fit within the cap dimensions and relative to the reinforcing steel.

18.1.7. Abutment Construction Joints

To accommodate normal construction practices, the bridge engineer should detail the following horizontal construction joints in the contract documents:

1. **Semi-Integral Abutments**. Place a mandatory construction joint between the approach slab and the top of the diaphragm.
2. **Seat Abutments**. Allow a horizontal construction joint between the top of the abutment seat and the bottom of the backwall. Some expansion joint types may require another construction joint at the approach slab seat.

Planned vertical construction joints are normally associated with staged construction. Make provisions for splicing or mechanical reinforcing couplers on horizontal reinforcing steel. Vertical reinforcing steel should be at least 3 inches from the construction joint.

Show keyways or roughened surfaces consistent with the structural design of the joint.

When the joint will be exposed to public view in the finished structure, provide a chamfered groove or similar technique to hide the joint. Allow a vertical construction joint between the wingwall-abutment interface.

18.1.8. Wingwalls

Reference: LRFD Article 11.6.1.4.

Provide wingwalls of sufficient length to retain the roadway embankment and to furnish protection against erosion. Figure 18-4 illustrates the typical dimensions and grading for wingwalls.

Orientation

Standard DOT&PF practice is to use wingwalls aligned parallel to the roadway centerline and attached to the abutment cap. The outside face of the wingwall should be co-linear with the bridge edge of deck. The bridge rails or barriers are supported by and extend to the end of the wingwall.

Occasionally, site constraints will require the use of wingwalls aligned parallel to the centerline of abutment bearing ("elephant ear" wingwalls). These wingwalls are susceptible to erosion which can result in undermining of approach rail posts and edge of roadway shoulder.

Thermal movement between the approach guardrail and bridge rails may also require special attention when elephant ear wingwalls are used. Only use "flared" wingwalls in combination with box culverts.

Length

Wingwall length is determined by extending the wingwall 5 feet to 8 feet beyond the hinge point between the embankment slope and the edge of shoulder.

Do not extend wingwalls more than 20 feet behind the rear face of the abutment without special design and detailing. Consider pile-supported, unattached, or other wingwall types for lengths greater than 20 feet.

Thickness

The thickness of any wingwall should match the barrier or curb width, but must not be less than 12 inches. Typical wingwall widths are 15 inches and 18 inches.

Unattached Wingwalls

Design unattached wingwalls as retaining walls. Unattached wingwalls are generally cast-in-place concrete retaining walls. Provide an expansion joint between the unattached wingwall and abutment. See Chapter 21 for DOT&PF practices on retaining walls.

18.1.9. Drainage

Provide positive drainage as needed in the embankment behind the abutment and wingwalls by using select backfill, porous backfill, weep holes, perforated drain pipe, a manufactured backwall drainage system, or a combination of these options. Provide details of the selected drainage system on the bridge plans.

Always consider ground water levels when evaluating an appropriate drainage system. Do not install drainage systems that allow pressurized backwater to saturate the abutment backfill during high-water events.

Generally, for relatively shallow girders supported on semi-integral abutments with parallel wingwalls, or elephant ear wingwalls less than 10 feet long, select backfill and porous backfill will be sufficient to promote good drainage.

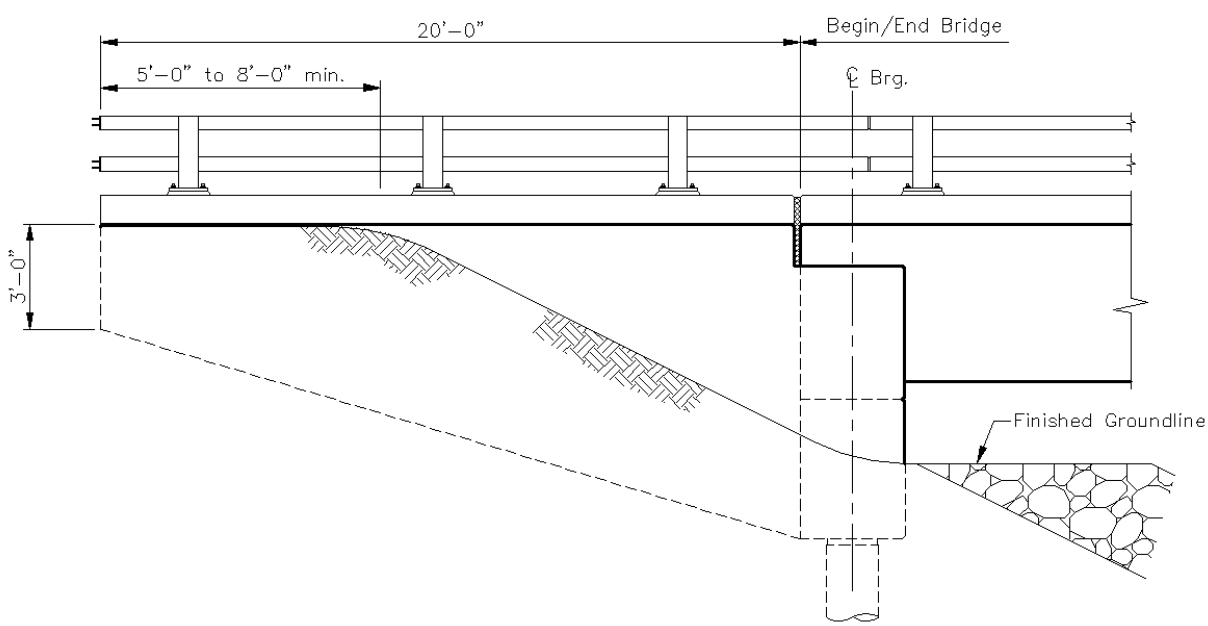


Figure 18-4
Typical Wingwall

18.1.10. Backfill

As discussed in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* Article C3.3, "Abutments as an Additional Energy-Dissipation Mechanism", the advantages of including abutments in the ERS could be offset by settlement of the fill during a seismic event. Typical DOT&PF practice does not include the abutments as part of the ERS but does require placement of Structural Fill behind the abutments to contribute to the resistance of seismic forces. Structural Fill is considered to be superior material with higher strength and stiffness than conventional roadway embankment material. For most bridges, the Structural Fill volume includes the width between the wingwalls, height from the top of superstructure or bottom of approach slab to the bottom of the abutment cap, and length of 50 feet from the beginning or end of bridges on paved roads and 30 feet for bridges on unpaved roads.

18.2. Piers

Reference: LRFD Article 11.7.

See Section 11.6.3 for DOT&PF practices on the selection of pier types.

18.2.1. Seismic Considerations

Design piers for non-seismic loads, then check them for seismic adequacy.

18.2.2. Pier Caps

Usage

In general, DOT&PF uses drop pier caps (non-integral with the superstructure) supported by pile extensions, a single column, multiple columns, or a solid pier wall. See Section 11.6.3 for more discussion.

Design

The cap depth-versus-length geometry affects the design of the pier caps. Where the distance between the centerline of the girder bearing and the column is less than approximately twice the depth of the cap, it may be appropriate to use the strut-and-tie model in LRFD Article 5.8.2 for the design of the cap; otherwise, use the sectional (beam) model for moment and shear.

Cap Width

Cap width is generally determined by adding 18 inches to the diameter of the supporting pipe piles. For caps supported on cast-in-place concrete columns or wall piers, the cap width should be 6 inches wider than the column diameter. Verify the resulting cap is sufficiently wide to accommodate the beam-seat widths dictated by seismic requirements.

Drop Caps

For crowned roadway sections, the bottom of the cap is level unless the bridge is very wide (greater than 100 feet). For superelevated cross sections, slope the bottom of the cap at the same rate as the cross slope of the top of the bridge deck. For decked bulb-tee girder bridges, slope the top of the cap parallel to the roadway crown or superelevation. Other girder types are set plumb. Thus, step down the tops of drop caps to account for elevation differences between girders with conventional cast-in-place decks.

18.2.3. Pier Cross Sections

The following summarizes DOT&PF practices for the cross section of piers.

1. **Round Columns**. The standard column has a minimum diameter of 2 feet with incremental increases in diameter of 6 inches. The preferred diameters for pile extension piers are 2 feet, 3 feet, and 4 feet.

2. **Solid Walls**. The minimum thickness is 2 feet (2′-6″ for railroad crash walls), which may be widened at the top to accommodate the bridge seat where required. Axial forces in the boundary edges of wall piers subjected to seismic loads may result in out-of-plane buckling, which may lead to excessive damage and loss of vertical load carrying capacity. Out-of-plane buckling of wall piers is affected by the wall pier geometry, reinforcement ratio, in-plane inelastic seismic demands and axial loading.

   The minimum thickness of wall piers, b_w, shall be taken as the greatest of the following:

   - 2.5 feet for railroad crash walls or 2 feet for all other applications

   - If the height of the wall, h_w, is greater than 1.5 times the length of the wall, l_w, (see Figure 18-6) then the minimum wall thickness, b_w, shall be determined from Figure 18-5

   In lieu of Figure 18-5, the refined analysis for out-of-plane buckling of wall piers from Haro, A.G., Kowalsky, M.J., and Chai, Y.H. (2017) may be used to determine the minimum wall thickness, but in no case shall the wall thickness be less than 2 feet or 2.5 feet for railroad crash walls.

   Wall piers may be provided with a cap beam if wider support width is required.

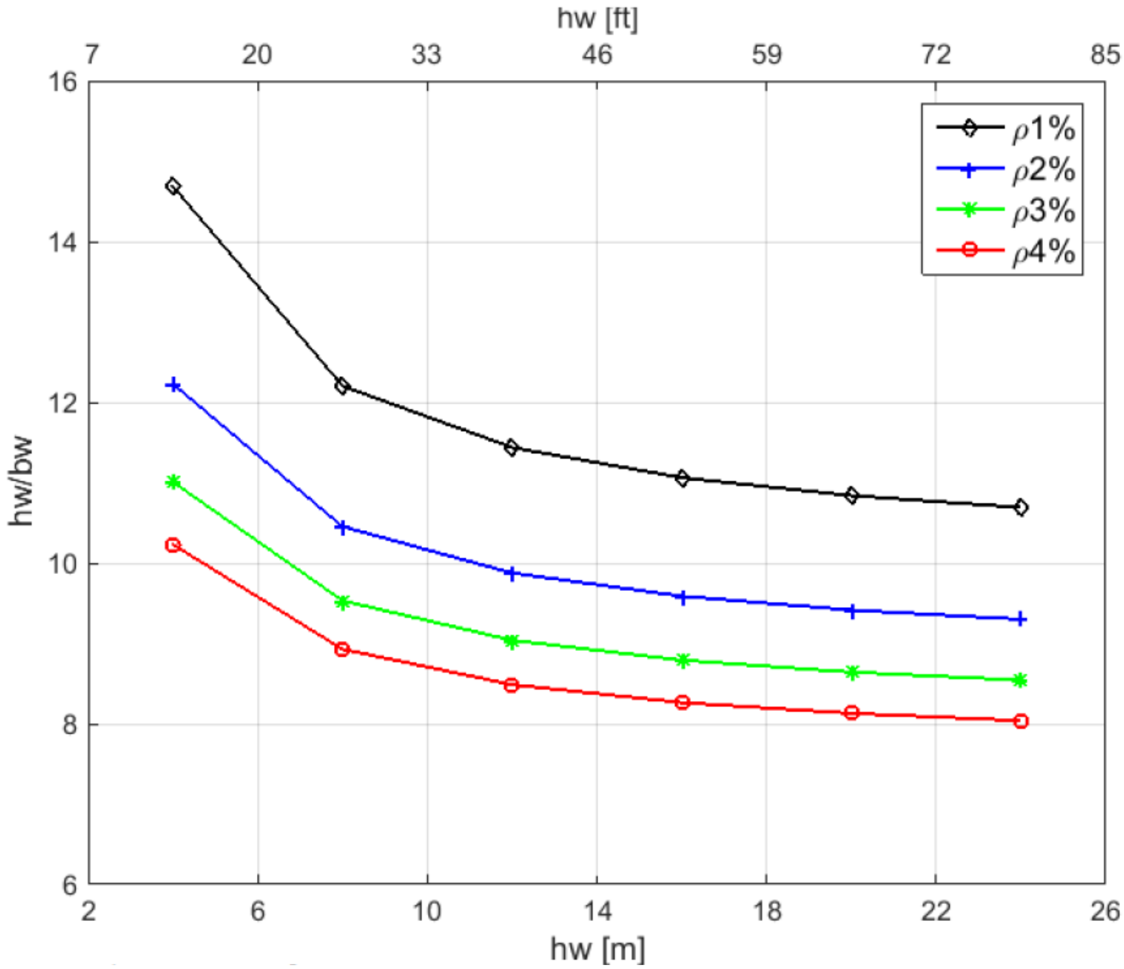


Figure 58-5
Minimum Wall Pier Thickness

18.2.4. Pier Foundations

Typical DOT&PF practice is to support single-column piers on oversized drilled shafts and to support pile bents on driven pipe piles. Multi-column or wall piers are infrequently used but may be considered where conditions warrant their use.

Enlarge the diameter of the drilled shaft relative to the column to force plastic hinging in the column and protect the drilled shaft from inelastic action. The drilled shaft diameter is typically 24 inches larger than the column diameter. Confirm that the diameters selected for the column and shaft will accommodate the overlapping reinforcing steel cages and cover requirements in both the column and drilled shaft. See Section 17.5 for a discussion on drilled shafts.

18.2.5. Column Reinforcement

Section 14.2 discusses DOT&PF practices for the reinforcement of structural concrete. This includes:

- concrete cover,
- bar spacing,
- lateral confinement reinforcement,
- corrosion protection,
- development of reinforcement, and
- splices.

The design of concrete pier columns must meet all applicable requirements in Section 14.2.

Transverse Reinforcement
Reference: LRFD Article 5.11.

General. Use spirals as transverse reinforcing steel in round columns. Allow butt-welded (electric "flash" resistance) spliced hoops in high seismic areas with radiographic testing and destructive testing.

Spiral Splices. Almost all spiral reinforcement will require a splice. LRFD Article 5.11 provides requirements for splices in spiral reinforcement. The contract documents must identify plastic hinge regions where a spiral splice is not allowed. Refer to Section 14.2.1.

Longitudinal Reinforcement
Reference: LRFD Article 5.11.

Use #8 or larger longitudinal column reinforcing bars, with #10 bars being the preferred minimum. Detail the longitudinal reinforcing steel continuous with a maximum spacing of 8 inches center-to-center.

Fully develop the longitudinal column reinforcing bars where these bars enter into the pier cap and the spread footing, pile cap, or drilled shaft. Longitudinal column reinforcing bars extend into the pier cap to be as close as possible to the top of the cap.

The preferred detail for longitudinal reinforcement is continuous, unspliced reinforcement. Provide a note on the bridge plans delineating the “no splice zones.”

If longitudinal column reinforcing bars require splices, use the provisions in LRFD Article 5.11. Do not locate splices within the plastic-hinge regions of the column. (Refer to Section 14.2.1.) Use a minimum stagger of 2 feet between adjacent splices. Also stagger splices in bundled bars at a minimum of 2 feet. If epoxy-coated bars are used, specify mechanical couplers tested with reinforcing bars coated as required for the design, and the couplers must use a compatible coating.

The contractor is not permitted to change the location or type of splice from those in the contract documents unless approved by the bridge engineer.

18.2.6. Column Construction Joints

Use construction joints at the top and bottom of the column. Where columns exceed 25 feet in height, permit intermediate construction joints. Where applicable, locate all construction joints at least 12 inches above the water elevation expected during construction.

18.2.7. Solid Walls

It is acceptable to reduce the dimensions of the wall in the transverse direction by providing cantilevers to form a hammerhead pier. Figure 18-6 illustrates the typical detailing for pier wall tie bars.

18.2.8. Pipe Pile Extension Bents

Pipe pile extension bents have proven to be constructable and cost-effective, and to provide reliable performance. They are the most commonly used pier type in Alaska. Use a single row of vertical piles. Battered piles are not allowed without approval of the Chief Bridge Engineer.

Filling steel pipe piles with concrete increases the member’s strength and stiffness. The concrete core also provides a means of connecting the pipe to the reinforced concrete cap beam. The moment capacity of a concrete-filled pipe pile is about three to four times that of a comparably sized reinforced concrete column.

The concrete-filled core in the pipe pile extends to a point where the moment demand is less than half of the maximum moment demand (i.e., below ground plastic hinge moment). The length of the concrete-filled core must include the effects of scour. Extend the concrete below the bottom of liquefiable soil layers into competent soil.

Extending the steel pipe pile into the concrete cap beam results in very large joint stresses and flexural demands that lead to unacceptable flexural hinging in the cap beam.

To limit the demands acting on the cap beam, the steel pipe portion of the pile is terminated several inches below the bottom of the cap beam. The resulting forces acting on the cap beam are those of the reinforced concrete core alone and the design follows the same procedure as that of a conventional reinforced concrete column-to-cap beam design.

The moment-curvature relationship of concrete-filled steel pipe piles can be calculated in the same manner as that used for conventional reinforced concrete (i.e. equilibrium and strain compatibility).

Use the expected material properties of the steel shell in the seismic analysis and design of concrete-filled steel pipe piles.

The expected material properties for commonly used steel pipe piles are provided in Table 18-1, in which D is the outside diameter of the pipe and t is the pipe wall thickness. For spiral welded pipe piles fabricated in accordance with the DOT&PF Special Provisions, use the properties of ASTM A709 Grade 50T3.

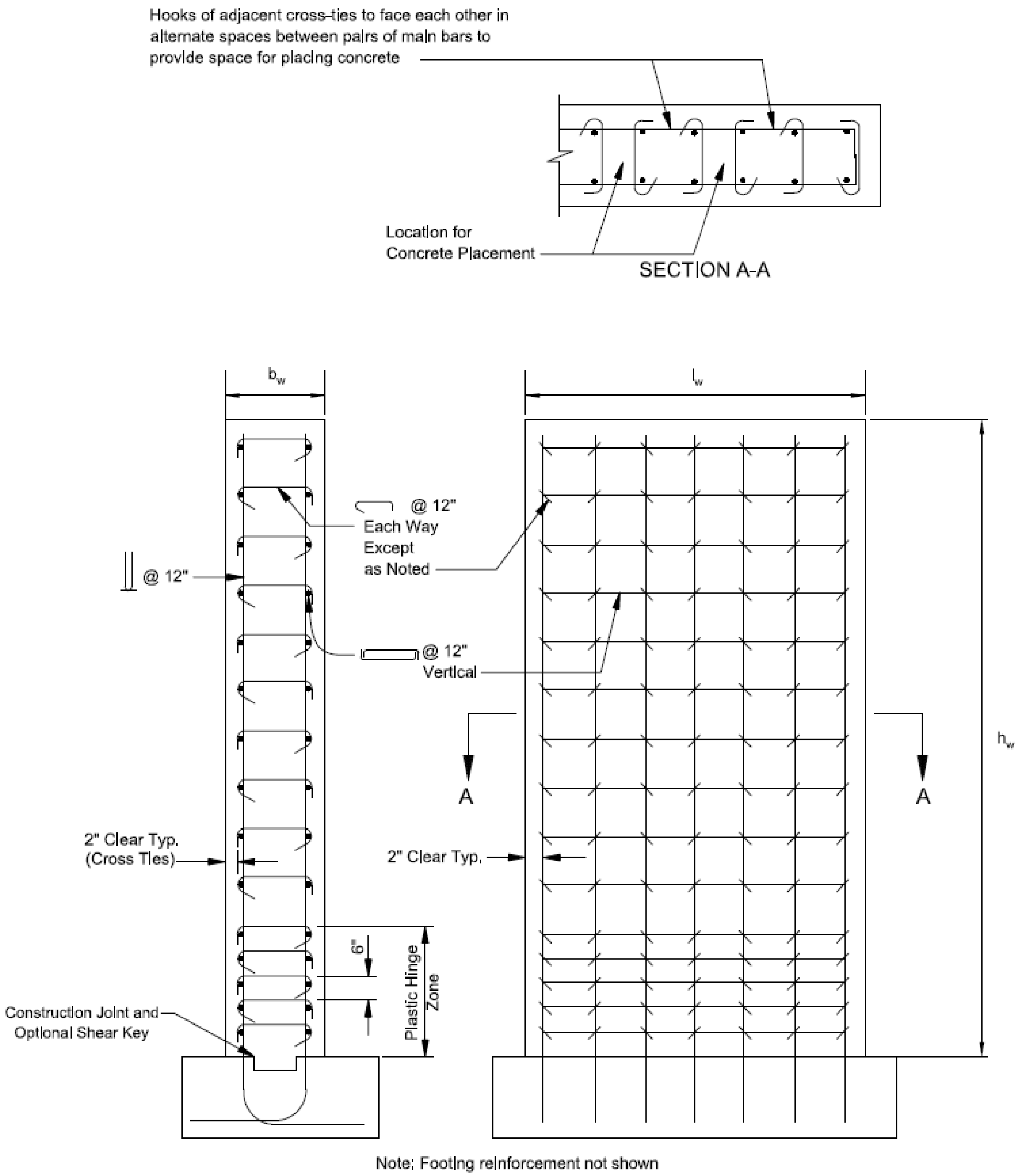


Figure 18-6
Pier Wall Tie Bars

The ultimate curvature of a concrete-filled pipe shall be based upon the reduced ultimate tensile strain of the steel pipe.

The onset of pipe wall buckling strain may be used to evaluate expected pipe performance under the design seismic event.

The Mander model can be used as the basis of generating the stress-strain relationship of the confined concrete core but the maximum confined concrete compressive strain shall not be taken greater than 0.02 (Brown at al 2013).

Table 18-1
Seismic Steel Pipe Pile Material Properties

Property	Notation	API 5L X52 PSL 2	ASTM A709 Grade 50T3	ASTM A53 Grade B
Specified minimum yield stress (ksi)	f_y	52.2	50	35
Expected yield stress (ksi)	f_{ye}	60	55	55
Expected tensile strength (ksi)	f_{ue}	78	78	78
Expected yield strain	ε_{ye}	0.0021	0.0019	0.0019
Onset of strain hardening	ε_{sh}	0.015	0.015	0.015
Onset of pipe wall buckling strain	ε_{cr}	0.022-(D/t)/9000	0.022-(D/t)/9000	0.022-(D/t)/9000
Reduced ultimate tensile strain	ε^{R}_{su}	0.026	0.026	0.026
Ultimate tensile strain	ε_{su}	0.12	0.12	0.09
Overstrength factor	λ_{mo}	1.2	1.2	1.4

18.3. Cold Climate Effects on Earthquake Resisting Elements

Much of Alaska experiences prolonged periods of temperatures below -40ºF. Concrete and steel demonstrate increasing strength at decreasing temperature. While not normally problematic for most bridge members, members that are sized based upon capacity design principles (i.e., capacity-protected elements) may experience increased demands at low temperatures.

Review the historic climate data at the bridge site. Include the effects of cold climate in the design if the record low temperature at the site is less than -20ºF.

If the adjoining capacity-protected member is insulated from severe temperature effects (e.g. buried footing or drilled shaft) then include the cold climate effects when determining the overstrength plastic hinging moment and associated forces of the hinging element. Analyze the moment-curvature response of the hinging elements using the following material properties (Montejo et al 2008):

$$f'_{ce\text{-}cold} = 1.4 \times f'_{ce}$$
$$f_{ye\text{-}cold} = 1.1 \times f_{ye}$$
$$f_{ue\text{-}cold} = 1.1 \times f_{ue}$$

where:

f'_{ce} = expected concrete compressive strength
f_{ye} = expected yield stress of steel
f_{ue} = expected tensile strength of steel

If both the hinging element and the capacity-protected element are exposed to the same temperature (e.g. column-cap connection) then the temperature related adjustments noted above would be expected to occur in both members.

Despite the increase in material strength, neither concrete nor reinforcing steel demonstrates a significant change in strain response at -40ºF. However a decrease in the analytical plastic hinge length occurs and is required to be taken as:

$$L_{p\text{-}cold} = 0.6 \times L_p$$

where:

L_p = analytical plastic hinge length

The effects of cold climate may also impact the stiffness of the supporting soils as outlined in Section 17.6.4.

18.4. References.

Haro, A.G., Kowalsky, M.J., and Chai, Y.H. (2017). *Seismic Load Path Effects in Reinforced Concrete Bridge Columns and Wall Piers – Volume 2: Out-of-Plane Buckling Instability of Pier Walls*. North Carolina State University, University of California Davis.

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19. Expansion Joints and Bearings

19.1. Expansion Joints

Reference: LRFD Articles 14.4 and 14.5.

19.1.1. General

Expansion joints accommodate the expansion and contraction of bridges due to temperature variations. The following general criteria apply to all expansion joints in bridges:

Minimize Number
When conditions permit, the bridge engineer should eliminate expansion joints and tie the approach slab into the superstructure. Where expansion joints are required to accommodate movement, minimize the number of joints.

Service Requirements
Failed joints contribute to many of the maintenance problems on bridges. Where joints are required, consider the long-term performance and maintenance requirements of the expansion joints.

Consistency of Joint Details
Where possible, use the same type of joint and joint construction details throughout the bridge.

Temperature Range
Use Procedure A of LRFD Article 3.12.1 as modified by replacing LRFD Table 3.12.2.1-1 with the temperature ranges in Table 19-1 specific to Alaska unless more detailed, site-specific information is available.

Construction at extreme temperatures (i.e., at or near the minimum or maximum temperatures of the assumed range) results in thermal movements in a single direction.

Recess Detail
Recess embedded steel elements, such as approach slab protection angles and strip seal expansion joint restrainers, 0.25 inch from finished grade. This recess provides protection from snowplow blades and accommodates milling of the concrete adjacent to the joints.

Effects of Skew
The thermal movements of skewed bridges are such that asymmetrical movements (“racking”) can occur along the length of the expansion joints. The movement is not solely in the longitudinal direction. The acute corners of a bridge with parallel skewed supports tend to expand and contract more than the obtuse corners, causing the joint to rack.

Racking should be limited to 20 percent of the rated movement of the joint. For bridges with expansion joints, avoid skews between 25 degrees and 35 degrees due to plow snagging.

Other Geometric Considerations
Horizontally curved bridges and bridges with other special geometric elements, such as splayed girders, do not necessarily expand and contract in the longitudinal direction of the girders.

A refined analysis of the entire bridge may be necessary to characterize the thermal movement of complex bridges. The effect of thermal movements on the joints of complex bridges could be more pronounced compared to bridges with simple geometrics.

The bridge engineer should use a refined analysis of horizontally curved, steel-girder bridges to estimate thermal effects, because even slight curvature may develop significant movements in the radial direction.

Table 19-1
Alaska Temperature Ranges

DOT&PF Region	Temperature Range (˚F)
Southcoast (maritime)	120
Central (intermediate)	140
Northern (interior/continental)	160

Blockouts
Provide details on the plans for blockouts in decks and approach slabs at expansion joints to allow for placement of the joint. During construction, contractors will install the expansion joint assembly and place the blockout concrete after profile grinding has been completed.

Cover Plates Over Expansion Joints
Use cover plates over expansion joints at sidewalks. Consider using cover plates in the shoulder area where bicycles are anticipated in the roadway. Use cover plates when strip seals are used. Overlap the cover plate consistent with the traffic direction to prevent snowplow snagging.

19.1.2. Expansion Joint Selection and Design

Reference: LRFD Article 14.5.3.2.

Table 19-2 presents the types of expansion joints used by DOT&PF and their maximum joint movement.

Select the type of expansion joint and its required movement rating based on the expansion and racking demands, skew, gap widths, and whether the joint is new or a retrofit.

Gap width is the perpendicular distance between the faces of the joint at the road surface. Use a minimum gap of not less than 1 inch for steel bridges. The gap for concrete bridges may be less than 1 inch where creep and shrinkage must be considered. Use a maximum gap width of 4 inches for strip seals and 3 inches for individual components of modular joints.

Silicone Joint Sealant
Reference: LRFD Article 14.5.6.5.

DOT&PF practice is to use this system where anticipated movements are small and where the strip seal joint is impractical.

The joint width at the time of installation dictates the movement capacity of this type of joint, which is a function of the installation width plus or minus some percent of original gap size.

The silicone joint sealant is easily maintained because local joint failures can be easily repaired.

This system can be bonded to concrete or steel, although bonding to steel is more effective in repairs and rehabilitation because the steel bonding surface is more easily cleaned.

Closed-Cell Compression Seal
Low-density, closed-cell foam products consist of pre-formed shapes compressed into the joint. For the size of the material and movement capacity, follow the manufacturer's recommendations.

Larger joints may also require a cover plate for protection of the compression seal. Closed-cell compression seals are used typically for rehabilitation.

Strip Seal
Reference: LRFD Article 14.5.6.7.

A strip seal consists of a gland rigidly attached to a steel restrainer on both sides of the joint. The material is pre-molded into a "V" shape that opens as the joint width increases and closes as the joint width decreases. Strip seal joints are usually protected by a steel cover plate. Strip seals are typically used for new construction.

Strip seal joints are watertight when properly installed. Under typical conditions, the life of a strip seal tends to be longer than that of other joint seals. However, these seals can be difficult to replace and splices in the gland should be avoided. Snowplows can damage the joint, especially if the skew is 20 degrees or greater.

Table 19-2
Expansion Joint Selection

Joint Type	Total Joint Movement
Silicone Joint Sealant	≤ 3 inches
Closed-Cell Compression Seal	≤ 4 inches
Strip Seal	≤ 4 inches
Modular Expansion	> 4 inches

Where practical and where additional protection for bearing assemblies and hinges is warranted, provide a secondary sealing system below the expansion joint assembly.

DOT&PF prefers strip seal glands made of natural rubber (virgin natural polyisoprene); however, as natural rubber becomes less available, the bridge engineer must consider using synthetic rubber (i.e., neoprene).

Strip seals and closed-cell compression foam have replaced preformed elastomeric joint seals on DOT&PF bridge designs. Elastomeric compression seals were difficult to compress and install in warm weather, and they frequently exceeded their tensile capacity in extremely cold weather. They are also not available in natural rubber.

Modular Expansion
Reference: LRFD Article 14.5.6.9

Due to their expense and maintenance requirements, use modular joints only where necessary to accommodate movements greater than 4 inches.

In the selection of modular joint systems, use only those that have been designed to facilitate the repair and replacement of components and that have been verified by long-term in-service performance.

Include a detailed description of the requirements for the modular joint system in the contract documents.

The following apply to the design of modular-type expansion joints:

1. **Joint Support**. The blockouts and supports needed for modular joint systems are large and require special attention when detailing. For modular joints supported from the top of the girder, provide a detail of the supporting device in the contract documents.

2. **Splices**. Where practical, modular joints should be full length with no field splices across the roadway width. If a field splice is required for staged construction of a cast-in-place bridge deck, space the support girders at a maximum of 2 feet from the splice location, which should be outside of the wheel path.

3. **Synthetic-Rubber Seal**. The synthetic-rubber seal, which is a strip seal gland in a modular joint, is one piece across the roadway width, regardless of stage construction considerations.

19.2. Bearings

19.2.1. General

Reference: LRFD Articles 14.4, 14.6, and 14.8.

Movements

Bridge bearings accommodate superstructure movements and transmit the loads to the substructure.

The consideration of movement is important for bearing design, which includes both translations and rotations. The sources of movement include initial camber or curvature, construction loads, misalignment, construction tolerances, settlement of supports, thermal effects, creep, shrinkage, seismic, and traffic loading.

Effect of Camber and Construction Procedures

The initial camber of bridge girders induces bearing rotation. Initial camber may cause a larger initial rotation on the bearing, but this rotation may decrease as the construction of the bridge progresses. Rotation due to camber and the initial construction tolerances are sometimes the largest component of the total bearing rotation.

Evaluate both the initial rotation and its short duration. At intermediate stages of construction, add deflections and rotations due to the progressive weight of the bridge elements and construction equipment to the effects of live load and temperature.

Also, consider the direction of loads, movements, and rotations, because it is inappropriate to simply add the absolute maximum magnitudes of these design requirements. Do not consider combinations of absolute maximums that cannot realistically occur.

In special cases, it may be economical to install the bearing with an initial offset, or to adjust the position of the bearing after construction has started to minimize the adverse effect of these temporary initial conditions.

Seismic Requirements

Reference: LRFD Articles 14.6.5 and 14.8.3.2.

Bearing selection and design must be consistent with the intended seismic response of the entire bridge system.

Steel-reinforced elastomeric bearing pads should provide adequate seismic performance for most bridges; however, they cannot be counted on to transmit seismic forces to the substructure.

Superstructure seismic forces will typically be transferred to the substructure using shear keys or dowels. In unique situations, restrainers, shock transmission units, or dampers may be considered.

Do not apply these provisions to seismic isolation bearings or structural fuse bearings.

Anchor Bolts

Although the *LRFD Specifications* require anchor bolts in various circumstances, design needs should dictate anchor bolt use. Do not rely on anchor bolts to transfer lateral seismic loads.

19.2.2. Bearing Types

Steel-reinforced elastomeric bearings are typically the first option for all new bridges. Bridges with large movements resulting in excessive bearing pad heights may require sliding surfaces.

In general, the bridge engineer should restrain vertical displacements, allow rotations to occur as freely as possible, and either accommodate or restrain horizontal displacements. Distribute the loads among the bearings in accordance with the superstructure analysis.

The bridge engineer may use sole plates for steel girders to distribute the load uniformly.

Steel-Reinforced Elastomeric Bearings

These bearings are usually the preferred low-cost option and require minimal maintenance.

Limit the height of steel-reinforced elastomeric bearings to 6 inches. Provide elastomeric fixed bearings with a horizontal restraint (typically, a row of dowels connecting the diaphragm to the cap beam) adequate for the full horizontal load.

Polytetrafluoroethylene (PTFE) Sliding Surfaces

Reference: LRFD Article 14.7.2.

Where horizontal movements result in steel-reinforced elastomeric bearing pads exceeding the 6-inch height limit, use a combination bearing. These bearings use a steel-reinforced elastomeric pad to accommodate rotation and a stainless steel plate/PTFE sliding surface to provide translational capability. See Appendix 19.A for a design procedure and example.

Seismic Isolation Bearings

There are various types of seismic isolation bearings, most of which are proprietary. See the AASHTO *Guide Specifications for Seismic Isolation Design* and the FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges* for detailed information. Isolation bearings increase the fundamental period of vibration of the bridge resulting in lower seismic forces. Although this period shift lowers the seismic forces, it increases the seismic displacements.

Isolation bearings also provide improved damping characteristics to limit the seismic displacement demands.

Consider cold climate behavior when selecting seismic isolation bearings; the preference is to use friction pendulum bearings. Friction pendulum bearings are proprietary and require sole-source procurement.

Chapter 23 discusses the use of seismic isolation bearings on bridge rehabilitation projects.

19.2.3. Design of Steel-Reinforced Elastomeric Bearings

Reference: LRFD Articles 14.7.5.

Steel-reinforced elastomeric bearings may become excessively large if they are designed for loads greater than approximately 650 kips. Although no limiting maximum design load is specified, the maximum practical load capacity of a steel-reinforced elastomeric bearing pad is approximately 750 kips. If the design loads exceed 650 kips, the bridge engineer should check with manufacturers for availability.

Design Method

Use the Method B procedure in the *LFRD Specifications* for the design of steel-reinforced elastomeric bearings. Method B requires additional acceptance testing.

Use 100 percent of the total movement range previously specified in Table 19-1 for the design of bearings. This practice assumes that the bearing is installed at the maximum or minimum design temperature. DOT&PF practice increases the LRFD design value of 65 percent of the total movement as specified in LRFD Article 14.7.5.3.2.

Orientation

Orient elastomeric pads and bearings so that the long side is parallel to the principal axis of rotation.

Holes in Elastomer

Do not use holes in steel-reinforced elastomeric bearings.

Edge Distance

For elastomeric pads and bearings resting directly on a concrete bridge seat, use 3 inches as the minimum edge distance from the edge of the pad to the edge of the concrete seat.

Dimensions

The minimum elastomeric bearing length or width shall be 6 inches. Provide a minimum of 0.25 inches of cover at the edges of the steel shims.

Make the internal elastomer layers not less than approximate 0.5 inches to accommodate adequate fabrication.

Elastomer

Use Grade 5, natural rubber for steel-reinforced elastomeric bearings. Indicate the bearing loads and elastomer grade in the contract documents.

Appendix 19.A
PTFE/Elastomeric Bearings

19.A.1 General

The basic PTFE/elastomeric bearing design consists of:

- PTFE disks sliding on stainless steel surfaces to accommodate the longitudinal movements, and
- elastomeric bearing pads to accommodate the rotational movements.

PTFE/elastomeric bearings are suitable for structures with moderate to large longitudinal translations, and relatively small rotations. This non-proprietary bearing is simple to design and fabricate. Good performance can be attained with careful attention to loading, rotation, and the physical properties/limitations of the manufacturing materials.

19.A.2 PTFE/Elastomeric Bearing Components

PTFE/elastomeric bearings consist of five basic components:

- sole plate,
- PTFE disk,
- intermediate plate,
- elastomeric bearing pad, and
- masonry plate.

The function of these components and the typical manufacturing materials are (see Figure 19-1):

- **Sole Plate**. Transfers superstructure loads to the bearing and provides a stainless steel sliding surface for superstructure translation. The sole plate is fabricated from A709, Gr. 36 steel and has a stainless steel surfacing. The stainless steel surface is bonded to the sole plate with epoxy resin and stainless steel cap screws or by perimeter welding.

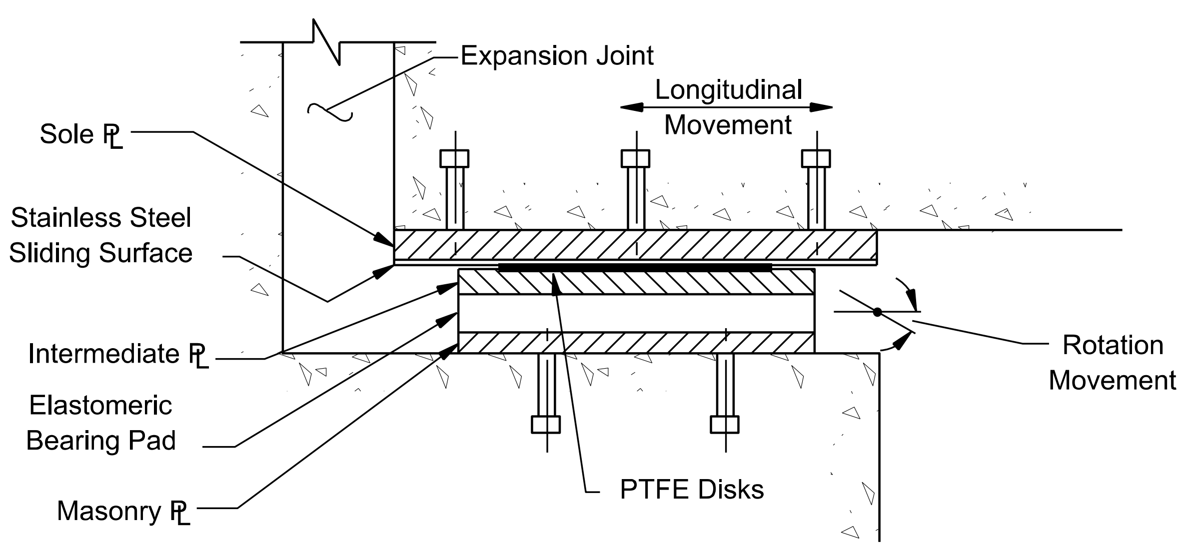


Figure 19-1
PTFE/Elastomeric Bearing

- **PTFE Disk**. Provides a low friction sliding surface for the sole plate. The PTFE disk is manufactured from 100 percent pure virgin unfilled dimpled sheet resin. The PTFE disk must be recessed one-half its thickness into the intermediate plate to control cold flow.

- **Intermediate Plate**. Transfers loads from PTFE disk to elastomeric pad. The intermediate plate is manufactured from A709, Gr. 36 steel.
- **Elastomeric Bearing Pad**. Allows rotation of the superstructure while maintaining 100 percent contact between the PTFE disk and the sole plate. The steel reinforced elastomeric bearing is fully vulcanized to the intermediate and masonry plates. Fabric reinforced pads are not allowed.
- **Masonry Plate**. The masonry plate transfers load from the elastomeric bearing pad and anchors the bearing to the seat. The masonry plate is fabricated from A709, Gr. 36 steel.

19.A.3 Design Requirements

PTFE/elastomeric bearings are designed in accordance with the *LRFD Specifications*, Sections 14 and 15. All loads are service loads. Minimum vertical loads are for dead loads and superimposed dead loads. Maximum vertical loads are for dead loads, superimposed dead loads, and live loads (no impact).

Unfilled PTFE sheet resin stresses are limited to 3,500 psi maximum. The design coefficient of friction varies from 0.08 to 0.04 at bearing pressures of 500 psi and 3,500 psi, respectively.

Steel reinforced elastomeric bearing pads with shape factors ≥ 7.5 may be loaded to a maximum stress of 1,000 psi. The shear modulus (G) used for design is 100 psi.

19.A.4 Design Guidelines

PTFE surfaces should be loaded to a minimum of 2,000 psi (DL only) for optimum performance. Use a design coefficient of friction of 0.06 for designs with bearing pressures from 2,000 psi to 3,500 psi. Actual lubricated friction values are lower; however, do not use these for design because the long-term effects of the grease are unknown. Use a minimum and maximum PTFE thickness of 3/16 inch and ¼ inch.

To reduce rotational stresses, orient rectangular bearings so that the long side is parallel to the axis about which the largest rotation occurs.

A bearing pad with a low shape factor accommodates rotation most readily, and a bearing pad with a high shape factor is best for resisting compression. Therefore, the best choice represents a compromise between the two. Use a minimum shape factor of 7.5.

19.A.5 Design Procedure

19.A.5.1 Elastomeric Bearing Pad

1. Determine the width (W) and length (L) of the elastomeric pad for the applied vertical load (DL + LL not including impact) using an allowable unit stress of 1,000 psi. The shape factor (S) ≥ 7.5:

$$\frac{\text{DL} + \text{LL}}{1{,}000\text{ psi}} = \text{W} \times \text{L}$$

2. Check the compressive strain of the elastomer due to dead load and live load from the stress/strain curves for various shape factors shown in Figure 19-2. These curves, developed by Caltrans, are based on tests of pads constructed with ½ inch layers of elastomers between steel plates meeting California specifications. To account for compressive creep of the elastomer under sustained dead load, the initial deflection from dead load is increased by 25 percent. The total deflection from dead load (DL) and live load (LL) shall not exceed 0.07 times the thickness of the elastomeric bearing.

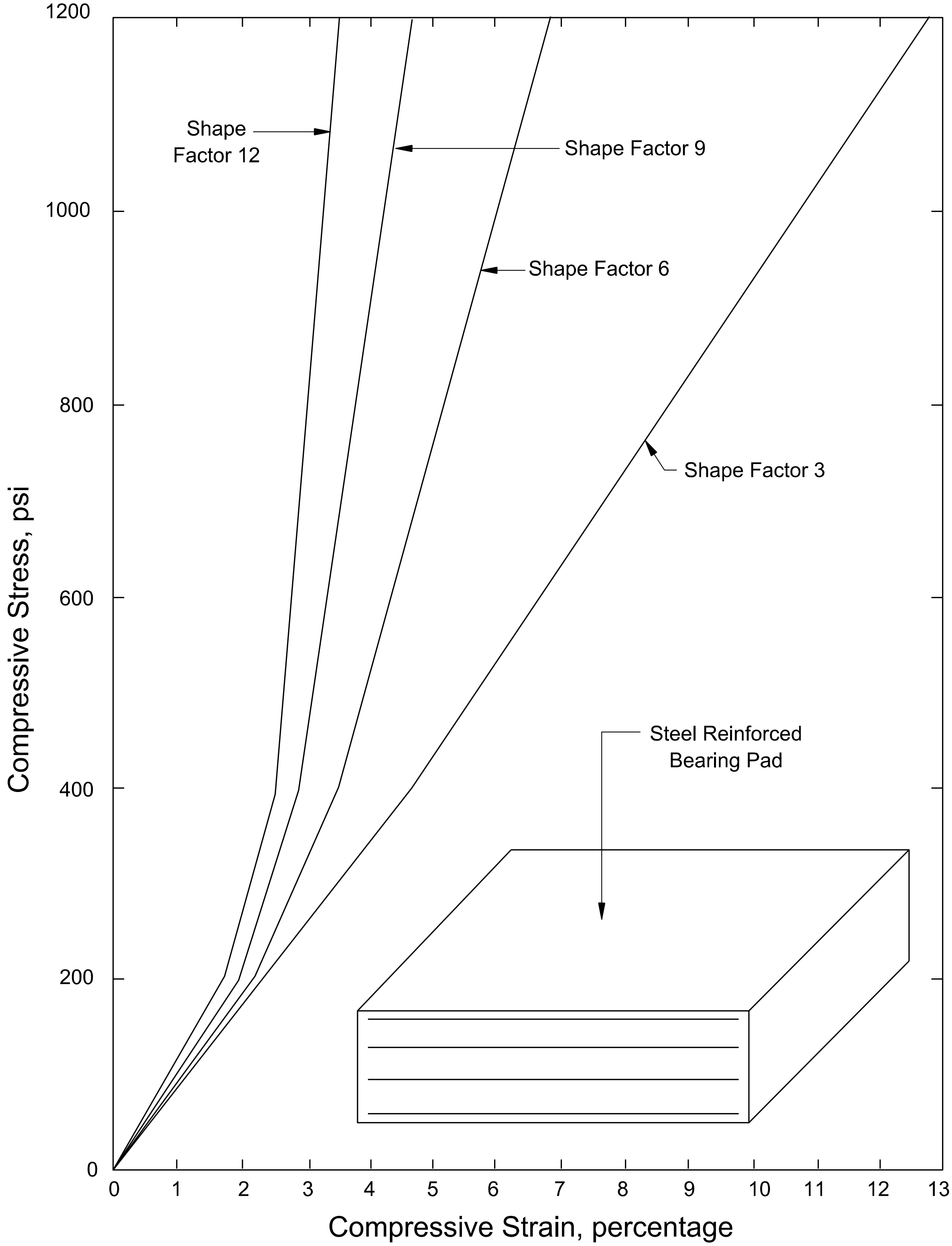


Figure 19-2
Recommended Compressive Stress vs. Strain Curves for Steel Reinforced Bearing Pads

3. Determine the initial thickness of the elastomer required for structure rotation. The structure rotation should include rotations from DL, LL, camber changes, construction tolerances, and erection sequences.

   The relative rotation between top and bottom surfaces of the bearing shall be limited by:

   $L\alpha_L + W\alpha_W \leq 2\Delta_c$ (for rectangular bearings)

   $D\sqrt{\alpha_L^2 + \alpha_W^2} \leq 2\Delta_c$ (for circular bearings)

   $$\Delta_c = \sum_i \varepsilon_{ci}\, t_i, \;\therefore\; \Delta_c = \varepsilon_{tot} T$$

   Therefore, the elastomer thickness (T) may be determined from:

   $$T \geq \frac{L\alpha_L + W\alpha_W}{2\varepsilon_{Tot}}$$ (rectangular bearings)

   $$T \geq \frac{D\sqrt{\alpha_L^2 + \alpha_W^2}}{2\varepsilon_{Tot}}$$ (circular bearings)

   Where:

   L = gross length of rectangular bearing parallel to the longitudinal axis of the bridge (inches)
   W = gross width of rectangular bearing perpendicular to longitudinal axis of the bridge (inches)
   D = gross diameter of circular bearing (inches)
   α_L, α_W = relative rotation of top and bottom surfaces of bearings about an axis perpendicular (parallel) to the longitudinal axis of the bridge (radians)
   Δ_c = instantaneous compressive deflection of bearing (inches)
   ε_{ci} = compressive strain of i^{th} elastomer layer (change in thickness divided by unstressed thickness)
   ε_{Tot} = total compressive strain of elastomer
   t_i = thickness of i^{th} elastomer layer (inches)
   T = total elastomer thickness of bearing (inches)
   = Σt_i

4. Determine the maximum allowable shear force (F_s) in the elastomer:

   $$F_s = G\frac{A}{T}\Delta_s$$

   Where:

   Δ_s = shear deflection of bearing (inches)
   G = shear modulus of elastomer (psi) at 73°F
   A = plan area of bearing (square inches)

   The maximum allowable shear force in the elastomer must be greater than the maximum lateral force required to slip the PTFE disk under dead load (see Figure 19-3).

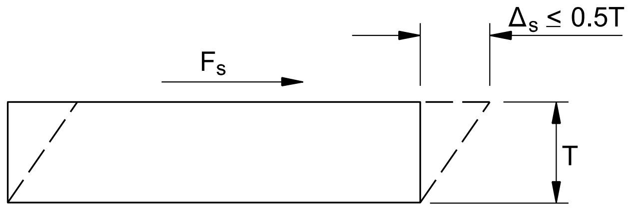


Figure 19-3
Shear Force in Elastomer

Note that the shear modulus (G) decreases with increasing temperature and increases with decreasing temperature. Values of 135 psi and 95 psi are recommended for this calculation.

The maximum shear deflection (Δ_s) in the elastomer shall be limited by:

$$T \geq 2\Delta_s$$

If the maximum allowable shear force is exceeded, the area of the elastomeric pad may be increased to provide greater shear capacity. It is evident from the above formulas that the elastomer design is sensitive to both the shear modulus and the friction force transmitted through the stainless steel sliding surface.

19.A.5.2 PTFE Disk

1. Determine the area of the PTFE disk required to support vertical loads (DL + LL, no impact), using a 3,500 psi maximum compressive stress. Note that the allowable compressive stress for the PTFE is 3.5 times the allowable stress for the elastomer. To minimize the thickness of the intermediate plate in which the PTFE is recessed, the length, width or radius of the PTFE should be such that the edge distance is held to a minimum. A 2,000 psi to 2,500 psi (DL only) design compressive stress on the PTFE should provide a reasonable intermediate top plate thickness.

   PTFE disks are used to facilitate fabrication of the recess in the steel intermediate plate.

2. Calculate the lateral force (F_f) required to slip the PTFE disk under dead load:

   F_f = μN
   μ = friction coefficient
   N = dead load

   Use the friction values given in LRFD Article 14.7.2.5. Note that the actual coefficient of friction will probably be less because the stainless steel slider plate will be coated with silicone grease.

3. Compare the maximum allowable shear force (F_s) in the elastomer with the lateral force (F_f) required to slip the PTFE under dead load:

   $$F_s \geq F_f$$

19.A.5.3 Intermediate Plate

1. Size the intermediate plate, length, and width to match the dimensions of the elastomeric bearing pad.

2. Determine required plate thickness. Design in accordance with AISC design procedure for column base plates. As a suggestion, convert PTFE disk area to equivalent square area to design plate thickness.

19.A.5.4 Sole Plate

Size the sole plate so that it remains in full contact with the PTFE disk under all loading conditions.

The safety overhang (L_o) provides a minimum edge distance and allows for additional sliding surface beyond the calculated movement. The value reflects the certainty or uncertainty of the total movement calculation:

D_D = diameter of PTFE disk

L_{max} = maximum longitudinal movement (including creep, shrinkage, post tensioning, thermal effects, and seismic).
T_m = maximum transverse movement
L_o = safety overhang

1. Longitudinal length (L_{sp}) and transverse width (W_{sp}) of sole plate:

   $$L_{sp} = D_D + L_{max}\,(\text{total}) + L_{oL}$$

   Single disk:

   $$W_{sp} = D_D + T_m + L_{oT}$$

2. Plate thickness (T_{sp}):

   Design in accordance with AISC design procedure for column base plates when mounted on concrete. Recommended minimum thickness is 0.75 inch.

3. Anchorage may be accomplished with shear studs, bolts, or welding depending on the structure type. Studs smaller than 0.75-inch diameter are not recommended.

19.A.5.5 Masonry Plate (When Needed)

1. Size the masonry plate, length, and width to match the dimensions of the elastomeric bearing pad unless a larger plate is required for anchorage.
2. A plate thickness of 0.75 inch is recommended.
3. Anchorage may be accomplished with shear studs, sleeved anchor bolts, or welding depending on the structure type. Studs smaller than 0.75 inch in diameter are not recommended.

19.A.6 Example — PTFE/Elastomeric Bearing, CIP P/S Structure

Given:	Structure Length	=	785 feet
	Contributory Length: 176 feet + 255 feet (CIP P/S)	=	431 feet
	DL Reaction/Girder: Service Load	=	271 kips
	LL Reaction/Girder: Service Load, No Impact	=	43 kips
	Moderate Temperature Zone: Rise and Fall	=	35°F
	f'_c = 4 ksi		
	F_y = 36 ksi		

19.A.6.1 Elastomeric Bearing Pad

1. Determine width (W) and length (L):

   $$\frac{DL + LL}{1{,}000\text{ psi}} = W \times L \qquad \frac{271\text{ kips} + 43\text{ kips}}{1{,}000\text{ psi}} = 314\text{ square inches}$$

   Try 12 inch × 28 inch Area = 336 square inches > 314 square inches Okay

   Note: Slender bearing pad selected to maximize rotation capacity.

   $$\text{Shape factor (S)} = \frac{12 \times 28}{12 + 28} = 8.4 > 7.5 \quad \therefore 1{,}000\text{ psi is okay}$$

   Actual load on elastomer:

$$\frac{271\text{ kips} + 43\text{ kips}}{12 \times 28} = 934\text{ psi (DL + LL)}$$

$$\frac{271\text{ kips}}{12 \times 28} = 807\text{ psi (DL)}$$

2. Check compressive strain:

   S = 8.4; obtain strain values from the curves shown on Figure 19-2.
   P_{TL} = 934 psi ε_{TL} = 4.4%
   P_{DL} = 807 psi ε_{DL} = 4.0%
   ε_{Total} = 4.4 + (4.0)(0.25) = 5.4% < 7.0% Okay

3. Determine initial thickness (T) for rotation:

   $$T \geq \frac{L\alpha_L + W\alpha_w}{2\varepsilon_{Tot}} \qquad \text{assume } W\alpha_w \text{ is negligible}$$

   $$\therefore T \geq \frac{L\alpha_L}{2\varepsilon_{Tot}} = \frac{(12)(0.015)}{2(0.054)} = 1.67\text{ inches; say 2 inches (elastomer only)}$$

   Structure Rotation: β_s = 0.003 radians
   Construction Rotation: β_c = 0.01 radians

   LRFD Specifications require 0.015 radians (minimum)

4. Determine the maximum allowable shear force (F_s) in the elastomer:

   $$F_s = G\frac{A}{T}\Delta_s = 100 \times \frac{12 \times 28}{2} \times 1 = 16.8\text{ kips}$$

   $$\Delta_{s\,max} = \frac{T}{2} = \frac{2}{2} = 1\text{ inch}$$

19.A.6.2 PTFE Disk

1. Determine area of PTFE disks:

   Use two disk design: -

   $$\text{DL area required} = \frac{271\text{ kips}}{2 \times 2{,}500\text{ psi}} = \frac{\pi D_D^2}{4} \qquad \therefore D_D = 8.3\text{-inch diameter}$$

   Try D_D = 8.5-inch diameter

   $$\text{DL stress on PTFE} = \frac{271\text{ kips}}{\left[2 \times \pi \times \frac{8.5^2}{4}\right]} = 2{,}388\text{ psi} > 2{,}000\text{ psi} \quad \text{Okay}$$

   $$\text{DL + LL on PTFE} = \frac{271 + 43}{\left[2 \times \pi \times \frac{8.5^2}{4}\right]} = 2{,}766\text{ psi} < 3{,}500\text{ psi} \quad \text{Okay}$$

   Use two ¼-inch × 8.5-inch diameter PTFE disks.

2. Calculate the lateral force (F_f) required to slip PTFE:

$F_f = \mu N$ $\mu = 0.06$
N = 271 kips (DL)
$F_f = 0.06 \times 271{,}000 = 16.2$ kips

3. Compare allowable shear force (F_s) required to slip force (F_F):

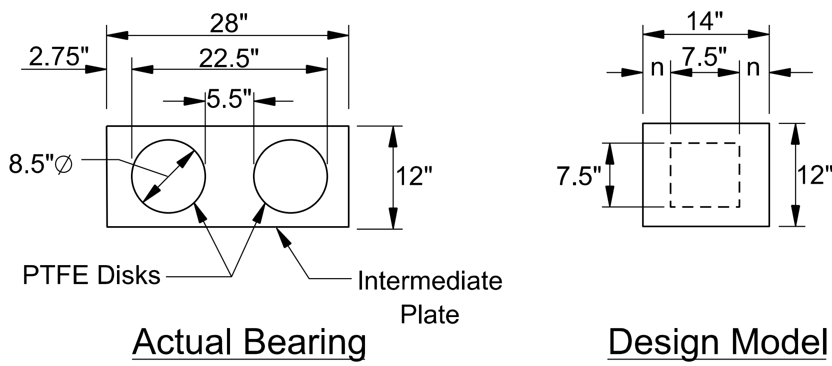
$F_s \geq F_f$ 16.8 > 16.2 kips Okay

19.A.6.3 Intermediate Plate

1. Size intermediate plate length (L) and width (W) to match elastomeric pad dimensions:

L = 12 inches
W = 28 inches

2. Determine plate thickness (T_p):



$$T_p = 2n\sqrt{\frac{f_p}{F_y}}$$

f_p = actual bearing pressure on elastomer
F_y = steel yield strength
F_b = $0.75F_y$

Model – Convert disk to equivalent square to determine (n):

$$f_p = \frac{271\text{ k} + 43\text{ k}}{12 \times 28} = 934\text{ psi}$$

$$n = \frac{14 - 7.5}{2} = 3.25\text{ inches}$$

$$T_p = 2 \times 3.25 \times \sqrt{\frac{934}{36{,}000}} = 1.05$$

Total plate thickness (T_p) including 1/8-inch recess:

T_p = 1.05 + 0.125 inch = 1.17 inch; say 1.25-inch thick

19.A.6.4 Sole Plate

1. Longitudinal length (L_{sp}) and width (W_{sp}):

   Temperature movement = (1.5) (2) (0.0000060) (35°F) (431 feet) (12 inch/foot) = ±3.26 inches

   P/S shortening = (0.70) (0.10 feet/100 feet) (431 feet) (12 inch/foot) = 3.62 inches

   Seismic movement = ±3.0 inches

   L_{max} = 3.26 + 3.62 + 2(3.0) = 12.88 inches
   L_{oL} = 2(1.0) (edge distance) = 2.0 inches
   D_D = 8.5 inches
   T_m = (seismic) = ± 1.0 inch
   L_{sp} = $D_D + L_{max}$ (total) + L_{oL}
   L_{sp} = 8.5 + 12.88 + 2 = 23.28 inches; say 24.0 inches
   W_{sp} = $D_D + T_m + L_o$
   L_{oT} = 2(1.0) (edge distance) = 2.0 inches
   W_{sp} = [(2)(8.5) + 5.5] + 2(1.0) + 2.0 = 26.5 inches

   Notes:

   (1) *Thermal movement was multiplied by 2 because 35°F is rise or fall temperature.*
   (2) *Thermal movement was multiplied by 1.5 because it is not always possible to place the sole plate at a "mean temperature."*
   (3) *Position sole plate to account for one directional movement of P/S shortening.*

2. Plate thickness (T_{sp}):

   Design in accordance with AISC design procedure for column base plates mounted on concrete:

   L_{sp} = 24.0 inches W_{sp} = 26.5 inches
   f'_c = 4 ksi F_y = 36 ksi

   $$f_b = 0.30 f'_c \sqrt{A_2 / A_1} \leq 0.60 f'_c$$

   Assume for this example that $A_2/A_1 = 1.5$

   Maximum bearing pressure (f_b) on loaded area:

   $$f_b = (0.30)(4{,}000)\sqrt{1.5} = 1{,}470 \text{ psi}$$

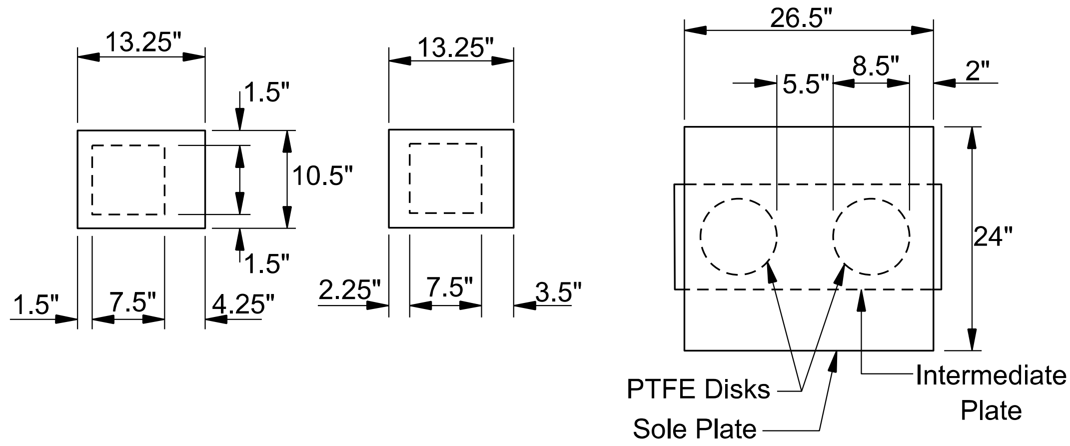
3. Determine required plate area:

   $$\frac{271 + 43}{1{,}470} = 213.6 \text{ square inches}$$

   Because the length of the sole plate was determined for sliding purposes, determine the required length to distribute the load to the concrete:

   $$\frac{213.6}{26.5} = 8.1 \text{ inches; use disk diameter + edge distance (8.5 + 2) = 10.5 inches}$$

   Design thickness for 26.5-inch × 10.5-inch plate



Design Model

Actual Bearing

Model information:

- Convert disks to equivalent square
- 1-inch transverse movement (T_m) shown

f_b = actual bearing pressure

F_y = steel yield strength (36 ksi)

F_b = 0.75 F_y

$$T_{sp} = 2n\sqrt{\frac{f_b}{F_y}}$$

$$f_b = \frac{271\text{ k} + 43\text{ k}}{2(10.5 \times 13.25)} = 1{,}128\text{ psi}$$

$$T_{sp} = 2 \times 4.25 \times \sqrt{\frac{1{,}128}{36{,}000}} = 1.50\text{ inches}$$

19.A.6.5 Masonry Plate

1. Size plate area to match elastomer area:

   Use 12-inch × 28-inch plate.

2. Plate thickness:

   A plate thickness of 0.75 inch is adequate because the masonry plate has the same area and load as the elastomeric pad:

   934 psi < 1,470 psi.

3. Anchorage:

   Because structure is cast in place, use shear studs.

19.A.6.6 Summary of Calculations

Plan Bearings:

- Elastomeric bearing pad 2″ × 1′-0″ × 2′-4″
- PTFE disk (2) ¼″ × 8½″ diameter
- Intermediate plate 1¼″ × 1′-0″ × 2′-4″
- Sole plate 1½″ × 2′-0″ × 2′-4″
- Masonry plate ¾″ × 1′-0″ × 2′-4″

Total Bearing Height:

- Sole plate 1.50
- Stainless steel 0.060
- PTFE disk (¼-inch thick recessed ⅛ inch) 0.125
- Intermediate plate 1.25
- Elastomer (2 + (4)(0.075)) 2.30
- Masonry plate 0.75
- Total 5.99 inches = 6.0 inches

20. Bridges in Remote Sites

20.1. Design Objectives

20.1.1. General

Chapter 20 presents practices that provide safe and cost-effective bridges in remote sites with low traffic volumes. Common characteristics of remote bridge locations to which this chapter applies are:

- They are off the main road system; barge service or air freight is frequently the primary means of transporting goods.
- Not on the NHS.
- No mining or other heavy industry is present.
- ADT $\leq$ 50 and ADTT $\leq$ 10.
- DOT&PF does not anticipate future traffic load or volume changes.

The DOT&PF typically provides the construction funding, selects the bridge type, and selects the appropriate standards for structural design. Designing and constructing bridges in remote sites presents special issues to consider, including:

- The bridge is often the only life-line between the village and the airport.
- The "heaviest vehicle in town" will provide useful information about the actual loading in these locations and should be identified as part of the design.
- The responsible local agency at remote sites does not typically have an engineer on staff or local design standards.
- There may be limited overweight vehicle enforcement in remote locations.
- The need to transport construction materials, equipment, and labor to remote sites can introduce design challenges.
- A bridge could have multiple users, such as pedestrians, all-terrain vehicles (ATVs), snow machines, etc.
- Balance between economy and durability as part of the life-cycle analysis offers long-term benefits. Due to access issues for remote sites, the goal is to provide a bridge that requires little or no maintenance. Life-cycle cost considerations justify higher initial costs, such as metallizing steel bridges to reduce maintenance costs.

20.1.2. Bridge Type Selection

DOT&PF typically uses steel girders or modular steel trusses in remote sites.

Treated timber bridges have historically demonstrated mixed results at some sites.

20.1.3. Safety

Reference: LRFD Articles C1.3.2.1 and C3.6.1.2.1.

In all cases, Chapter 20 intends to maintain the inherent safety of the *LRFD Specifications*. The HL-93 notional design load is based upon grandfathered load exemptions, which yields a realistic live-load model despite the lack of truck-traffic data for remote sites.

The statistics of load (mean and coefficient of variation) taken from freeway-type highways used for the calibration of the *LRFD Specifications* are not always appropriate for remote sites, allowing a slight modification of this approach.

The bridge engineer will apply the live-load load factors of LRFD Table 3.4.1-1, as modified herein, to the design of bridges in remote sites. The modifications to the load factors in Section 20.2.1.2 address lower ADTT, not the database of the original calibration.

20.1.4. Constructability/Maintainability/ Durability

Mobilization costs can dominate decision-making when constructing a bridge at a remote site. Neither roadway nor water access may be available. In addition, consider which materials to transport to the site; the objective is to specify materials that are extremely durable to maximize the probability of achieving a 75-year design life with little or no maintenance or rehabilitation. Follow these guidelines for remote bridge designs:

1. Consider what construction labor, materials, and equipment are not readily available at the site.

   Usually, the Contractor must transport all construction materials and equipment to these sites, typically over great distances. In addition, the Contractor may have to transport housing, vehicles, and other facilities for workers.

2. Avoid cast-in-place concrete components at remote sites where concrete batching is impractical. Use prefabricated/precast elements where possible (e.g., deck, caps, girders). Also consider using prefabricated abutments.

3. Design and detail prefabricated components considering the means of transportation to the site, possibly including cargo aircraft.

20.1.5. One-Lane vs. Two-Lane Bridge

For bridges at remote sites with an ADT less than 100 vehicles per day, one-lane bridges on two-lane, two-way roads can be considered during the Bridge Type Selection process. The roadway designer and regional traffic engineer must determine that a one-lane bridge can operate safely.

One-lane bridges should have pull-offs at each end where drivers can wait for oncoming traffic to clear. The minimum width of a one-lane bridge is 15 feet. Avoid using one-lane bridges wider than 16 feet because drivers may attempt to use them as two-lane structures.

Design one-lane bridges to accommodate future widening. For example, widenings that require cofferdams below the superstructure may be impractical, whereas pipe pile extension bents can be widened with greater ease and less cost.

Bridge Length

The longer the bridge, the more likely that an "undesirable" result would occur (i.e., two vehicles moving in the opposite direction meeting on the bridge). However, the longer the bridge, the higher the potential cost savings to constructing a single-lane bridge. This savings is not proportionate to the reduction in deck area. For example, a 50 percent reduction in the square footage of the deck may yield a 25 percent reduction in the total cost.

Sight Distance

If two vehicles on either end of the bridge cannot see one another, this is a disadvantage to installing a one-lane bridge on a two-way roadway.

Truck Traffic

With lower truck traffic on the roadway, a one-lane bridge is more feasible.

20.2. Limit States

20.2.1. Strength

Load Modifiers

Reference: LRFD Articles 1.3.2.1, 1.3.3, 1.3.4, and 1.3.5.

For all Strength limit states for bridges in remote sites, use:

$$\eta_D = \eta_R = 1.0$$

and

$$\eta_I = 0.95$$

yielding a load modifier for application in LRFD Equation 1.3.2.1-1 of:

$$\eta_i = 0.95$$

This load modifier represents the use of less stringent Strength limit-state criteria for bridges in remote sites. For all other limit states of bridges in remote sites, use a load modifier of:

$$\eta_i = 1.0$$

Live-Load Load-Factor Reduction

References: LRFD Articles 3.4.1 and C3.6.1.1.2.

The following applies:

1. For $50 \leq \text{ADTT} \leq 500$, reduce the live-load load factors in LRFD Table 3.4.1-1 to 95 percent of those specified.
2. For ADTT < 50, reduce the live-load load factors in LRFD Table 3.4.1-1 to 90 percent of those specified.

These reductions reflect the reduced probability of the design event occurring during a 75-year design life with reduced truck-traffic volume.

20.2.2. Service Live-Load Deflection Check

References: LRFD Articles 2.5.2.6.2, 2.5.2.6.3 and 3.6.1.3.2.

Do not apply the criteria for live-load deflection in LRFD Articles 2.5.2.6.2 and 3.6.1.3.2 or the criteria for span-to-depth ratios of LRFD Article 2.5.2.6.3 for the design of bridges in remote locations.

20.2.3. Load-Induced Finite-Life Fatigue II Limit State

References: LRFD Articles 3.4.1 and 6.6.1.2.

Design steel details on bridges in remote sites to provide the finite fatigue resistance associated with the ADTT, as specified in LRFD Equation 6.6.1.2.5-2, using the Fatigue II load combination of LRFD Table 3.4.1-1.

The ADTTs in remote sites are sufficiently low that the finite-life fatigue resistance is more appropriate as suggested in LRFD Table 6.6.1.2.3-2, where all of the governing ADTT thresholds for infinite-life are greater than 500 trucks per day.

20.3. Live-Load Analysis

Reference: LRFD Article 3.6.1.3, 4.6.2.2 and 4.6.3.

The following applies:

1. Apply the single-lane live-load distribution factors of LRFD Article 4.6.2.2 for the design of bridges in remote sites when using approximate methods of analysis. Do not apply the distribution factors for multiple lanes.

2. Where refined methods of analysis are used for the design of bridges in remote sites, load the model with only a single lane. The single-lane multiple presence factor of 1.2 from LRFD Table 3.6.1.1.2-1 still applies.

3. For determining negative moment in continuous spans, and the reaction at interior piers on bridges in remote sites, load all spans with a single design vehicle superimposed upon the design lane load.

   Do *not* apply 90 percent of the effect of two design trucks spaced a minimum of 50 feet between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load as specified in LRFD Article 3.6.1.3.

Although heavy vehicles as represented by the HL-93 notional live-load model may cross bridges in remote sites, it is highly unlikely that multiple heavy vehicles, transversely or longitudinally, will be on the bridge simultaneously.

As specified in Section 20.1.5, design one-lane bridges to accommodate future widening without difficulty. Therefore, the live-load analysis should consider this possibility in proportioning superstructure and substructure components for one-lane bridges.

20.4. Design Details

20.4.1. Metalizing or Galvanizing of Steel Bridges

Metalize or galvanize all steel bridges to be erected in remote sites to provide a relatively maintenance-free coating. Metalize field sections greater than 50 feet in length. Use hot-dip, galvanized field sections less than 50 feet in length. The 50-foot demarcation represents the length of commonly available galvanizing tanks.

20.4.2. Approach Slabs

Reference: LRFD Article 3.11.6.4.

Do not use approach slabs on bridges with an ADT less than 100 and an ADTT less than 50. In such cases, apply the full live-load surcharge (LS) of LRFD Table 3.11.6.4-1 to the abutments.

20.4.3. Bridge Rail Systems

Comply with the requirements in Section 16.5.

20.4.4. Future Wearing Surface

Design all bridges for a future wearing or gravel surface assuming an additional dead load of 50 psf over the deck area.

20.4.5. Foundations

Although equipment for pile driving adds to the cost, the crane may be used for other purposes anyway. The bridge engineer may consider the use of shallow foundations for one-span bridges; however, the relatively high cost of riprap and the difficulty to maintain the riprap may offset the potential cost savings. If piers are used, then deep foundations will likely be required at the abutments also.

20.4.6. Temporary (Detour) Bridges

Consider reduced design standards for temporary bridges. Reductions might include one-lane traffic, design live loads, non-crash tested bridge rails, hydraulic capacity consistent with the anticipated life of the temporary structure, etc. Document the decision(s) to reduce design standards in the project files.

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21. Miscellaneous Structural Elements

In general, the Bridge Section is responsible for all structural design elements for DOT&PF projects. Although the majority of the Section's work is for highway bridges, the transportation system includes a wide variety of other structural elements.

Sections 11 and 12 of the AASHTO *LRFD Specifications* discuss design and detailing requirements for retaining walls and buried structures.

This chapter presents DOT&PF policies, practices, and criteria for the structural design of these miscellaneous structural elements.

21.1. Retaining Walls

See Section 1120.4 in the *Alaska Highway Preconstruction Manual* for more discussion on retaining walls.

21.1.1. Applications

DOT&PF uses retaining walls to provide lateral support for a variety of applications:

- cuts in slopes for roadway alignments
- roadway widening where right-of-way is limited
- grade separations
- surcharge loads from adjacent buildings, highways, and other structures that must remain in place
- slope stabilization
- protection of environmentally sensitive areas
- excavation support

21.1.2. DOT&PF Classifications

Retaining walls are classified as follows:

1. **State-Designed Structures**. These walls are designed entirely by the DOT&PF or a consultant without the use of proprietary systems. Examples of this type of structure are cast-in-place concrete cantilever retaining walls, soldier-pile lagging walls, and steel sheet pile walls.

2. **Pre-Approved Proprietary Structures**. These walls are patented structures. The special provisions of the contract documents may list pre-approved proprietary structures as an alternative wall system based upon the recommendation of the Statewide Materials Section. Typically, these are Mechanically Stabilized Earth (MSE) retaining walls. For the use of proprietary walls on federal-aid projects, the DOT&PF must adhere to the Code of Federal Regulations, Title 23, Section 635.411 "Material or Product Selection."

21.1.3. Retaining Wall Selection and Design Process

Foundation Investigation and Report

The project manager will notify the foundation engineer of the need to conduct the foundation investigation for the wall and to prepare the Structural Foundation Engineering Report (SFER).

This report should describe soil conditions, make foundation engineering design recommendations, and identify and recommend feasible wall systems. Section 17.2 discusses the SFER in detail.

For retaining walls, the foundation engineer:

- performs the geotechnical investigations;
- provides recommendations for feasible wall types;
- recommends the nominal soil bearing and lateral earth design coefficients for gravity, surcharge, and seismic loading;
- performs the global and external stability checks; and
- determines if there is a need for special drainage features due to the selected wall type, site conditions, or both.

The foundation engineer will also provide the following information to the bridge engineer:

- earth pressure coefficients (e.g., k_a, k_o, k_p) and an estimate of the amount of deformation to develop the active and passive earth pressures
- unit weight of the backfill material
- allowable interface friction between cast-in-place and precast concrete footings and soil

- allowable bearing capacity
- expected settlement
- requirements for drainage control
- special construction requirements

Retaining Wall Type Selection

The selection of a wall type depends on several performance variables:

1. **Material Availability and Cost**. These are important considerations for every site. Mechanically Stabilized Earth walls usually require a select backfill material. These materials are not locally available in certain areas of the state.

   At remote sites (see Chapter 20), concrete may not be practical, and the necessary aggregate may not be available locally. Therefore, at remote sites, transportation costs for construction equipment and materials are a major consideration.

2. **Ease of Construction**. Evaluate the equipment requirements to construct a wall. Determine if the required equipment can be mobilized to the construction site and will have sufficient maneuvering room.

   Generally, a contractor can construct MSE and anchored walls with smaller tools and lifting equipment.

   For all wall types, some earth-moving equipment is required, but a tie-back wall minimizes the need.

3. **Cut vs. Fill Walls**. Some wall construction types are better suited in earth cuts and some in fills. For example, MSE walls perform well in new embankment but may require extra excavation, backfill, and right-of-way when used to retain soils adjacent to a depressed roadway.

4. **Drainage**. Retaining wall design forces typically assume fully drained conditions. All retaining wall designs shall address drainage and prevent the build-up of hydrostatic pressure.

5. **Settlement**. Potential settlement is also a consideration. Rigid walls do not tolerate settlement well. If significant settlement is anticipated, the most favorable walls are MSE walls.

   With limited construction space, a sheet pile wall or a tie-back soldier-pile wall may be ideal. The bridge engineer can use piling to resist settlement on cast-in-place concrete walls, but this solution is usually costly.

6. **Service Life**. Use of metal products in corrosive environments (e.g., marine, acidic soils) requires special attention. Always treat timber products with a preservative for ground contact and minimize the number of field cuts. Provide corrosion protection systems for the steel reinforcing bars in concrete exposed to salt.

7. **Surcharge**. Surcharge loading (loads along the top of the retained embankment) may require walls with additional strength and stiffness. Most structures built on top of retained embankments are sensitive to settlement.

8. **Aesthetics**. The aesthetic value of wall facings are important where visual exposure will be high, such as in urban settings.

9. **Railings**. All walls with an exposed face height exceeding 3 feet need a cable safety railing if they do not already have another type of railing or barrier.

Structural Design of CIP Concrete Retaining Walls

Standard Drawings are available for CIP concrete cantilever retaining walls. For non-standard walls, the Bridge Section, or a consultant experienced in wall design, will design all cast-in-place walls over 4 feet in height. Design the structural aspects of a wall using this *Manual* and Section 11 of the *LRFD Specifications*. The following summarizes the objectives:

1. **Design**. The bridge engineer will perform the internal stability design for the wall (e.g., wall dimensions and reinforcing configurations) and perform the overturning, sliding and bearing checks, using the geotechnical parameters provided by the foundation engineer.

2. **Contract Plans**. The bridge engineer and highway design engineer will coordinate to provide all construction details for the wall, including:

a. beginning and end of wall stations
b. horizontal wall alignment
c. elevation on top of wall at the beginning and end of wall, all profile break points, and roadway profile data at wall line
d. original and proposed profiles in front of and behind the retaining wall, including the ground line at wall ends
e. typical sections
f. elevation of highest permissible level for foundation construction (place the top of footings at least 2 feet below the frost line), location, depth, and extent of any unsuitable material to be removed and replaced
g. right-of-way limits shown on the plans
h. design high water and normal water levels at stream locations
i. quantities table showing estimated square feet of wall area and quantity of appurtenances and traffic barriers
j. the location of new or existing utilities in proximity of the retaining wall
k. reinforcing details and cross sectional dimensions
l. construction sequence requirements if applicable, including traffic control, access, and stage construction sequences
m. boring logs (provide the SFER as supplemental information to the contract bidding documents)
n. details of wall appurtenances (e.g., traffic barrier, coping, handrail, noise barrier, drainage outlets, location, configuration of signs and lighting including conduit location)

Design of Proprietary Retaining Walls

Provide sufficient geometric controls in the contract plans so that a vendor may prepare a system structural design. Typically, this is a line in the contract plans that provide the lateral offset and termini of the wall. The contract documents will identify those vendors that are qualified, either through a reference to the Qualified Products List or in the special provisions.

21.1.4. Retaining Wall Types

Walls are classified according to their construction method and the mechanism used to develop lateral support:

Construction Method

This may be either a “fill-wall” construction or “cut-wall” construction.

Fill-wall construction is where the wall is constructed from the base of the wall to the top (i.e., “bottom-up” construction).

Cut-wall construction is where the wall is constructed from the top of the wall to the base (i.e., “top-down” construction).

Note that the “cut” and “fill” designations refer to how the wall is constructed, not the nature of the earthwork (i.e., cut or fill) associated with the project.

Lateral Load Support

The basic mechanism of lateral load support may be either “externally stabilized” or “internally stabilized.”

Externally stabilized wall systems, such as a CIP concrete cantilever wall, use an external structural wall, against which stabilizing forces are mobilized.

Internally stabilized wall systems, such as an MSE wall, employ reinforcement that extends within and beyond the potential failure mass.

21.1.5. Fill Walls

MSE Walls

MSE walls are constructed using reinforced layers of earth fill with non-extensible (metallic) reinforcing. The facing for the walls shall be concrete panels. Advantages of MSE walls include:

- They tolerate larger settlements than a CIP concrete cantilever wall.
- They are relatively fast to build.
- They are relatively low in cost.

MSE walls are not allowed along bodies of water. See Section 21.2 for more information on MSE walls.

CIP Concrete Cantilever Walls

CIP concrete cantilever walls are best suited for sites characterized by good bearing material where minimal long-term settlement is anticipated. In some cases, weaker surface soils can be excavated and replaced with 5 feet of compacted gravel backfill to economically improve the bearing capacity.

In particularly, soft soils or when settlement may be a problem, walls can be pile supported. Piles add to the cost, especially relative to a MSE wall; however, for short wall lengths, the pile-supported CIP concrete cantilever wall may be a cost-effective option.

An important advantage of CIP walls is that they do not require special construction equipment, wall components, or specialty contractors. They can be up to 30 feet in height, although most are less than 20 feet in height. The footing width for these walls is normally ½ to ⅔ the wall height.

CIP concrete cantilever walls can be used in cut slope locations. In this case, the slope behind the face of the wall requires excavation to provide clearance for the construction of the wall footing. Do not use excavation slopes steeper than 1.5H:1V, which would result in significant excavations in sloped areas. In this case, a shored excavation may be required, or alternative wall types (e.g., soldier-pile walls) may be more suitable.

21.1.6. Cut Walls

Soldier-Pile Walls

Soldier pile walls involve installing H-piles every 6 to 10 feet and spanning the space between the H-piles with lagging. The H-piles are usually installed by grouting the H-pile into a drilled hole; however, they can also be installed by driving. The advantage of drilling is that this avoids vibrations and the potential for driving refusal.

The embedded depth of the soldier pile is approximately two times the exposed height. The exposed height is typically up to 15 feet. Lagging can be either treated timber or concrete panels.

A concrete facing may be cast in front of the soldier piles and lagging after the wall is at full height to improve aesthetics.

Anchored Walls

Ground-anchored wall systems (also called tie-back walls) typically consist of tensioned ground anchors connected to a concrete wall facing. Use ground anchors to construct soldier pile walls of a taller height. Ground anchors consist of a high-strength steel bar or prestressing strand that is grouted into an inclined borehole and then tensioned to provide a reaction force at the wall face. Anchors are typically laid out at 8-feet to 10-feet horizontal and vertical spacing, depending on the required anchor capacity. Each anchor is proof tested to confirm its capacity.

Anchored walls require specialized equipment to install and test, resulting in a higher cost relative to conventional walls. Consider the subsurface easement requirements for the anchoring system. The upper row of anchors can extend a distance behind the wall equal to the wall height plus up to 40 feet.

Soil-Nail Walls

A soil-nail wall uses top-down construction. The typical construction method includes:

- a vertical cut of approximately 4 feet;
- drill, insert, and grout soil nails;
- shotcrete exposed cut surface;
- repeat operation until total height of wall is complete; and
- for permanent applications, cast a reinforced concrete wall over the entire surface.

A soil-nail wall involves grouting large diameter rebar (e.g., #10 or larger) or strand into the soil at 4-feet to 6-feet spacing vertically and horizontally. The length of the rebar or strand will typically range from 0.7 times the wall height to 1.0 times the wall height, or more.

Soil-nail walls require specialty contractors. They can be difficult to construct in certain soil and groundwater conditions. For example, where seeps occur within the wall profile or in relatively clean sands and gravels, the soil may not stand at an exposed height for a sufficient time to install nails and apply shotcrete.

Sheet-Pile Walls

Sheet-piles are normally driven or vibrated into the ground with a pile driving hammer and are most suitable at sites where driving conditions are amenable to pile driving. Therefore, the bridge engineer must perform a driveability analysis. The SFER should address drivability when sheet-pile walls are considered. Sites with shallow rock or consisting of significant amounts of cobbles, boulders, or permafrost are not suitable for sheet-pile driving.

Generally, the sheet pile must be driven to a depth of two times the exposed height to meet stability requirements. Most sheet-pile walls are 10 feet to 15 feet or less in height. Although higher walls are possible, the structural design and installation requirements increase significantly. Taller sheet pile walls are possible, but require ground anchors that are typically attached to a horizontal waler beam installed across the face of the sheet piles.

21.1.7. References

Further guidance on walls:

- *Geotechnical Engineering Circular No. 2 – Earth Retaining Systems*, FHWA-SA-96-038
- *Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchored Systems*, FHWA-IF-99-015
- *Geotechnical Engineering Circular No. 7 – Soil Nail Walls*, FHWA-IF-03-017
- *Training Course in Earth Retaining Structures*, FHWA-NHI-132036
- *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines*, FHWA-SA-96-071

21.2. Mechanically Stabilized Earth (MSE) Walls

21.2.1. Responsibilities

DOT&PF

DOT&PF will conduct the global stability analysis and the external stability analyses with respect to sliding, overturning, slope stability, and bearing pressure failures. The contract documents identify the following typical information and details:

- location and elevation of leveling pad;
- total height of wall from top of leveling pad to top of coping/bottom of barrier rail;
- minimum 4-foot wide bench in front of walls located on top of slopes;
- no steeper than 2H:1V slopes in front of or on top of walls;
- strength properties of soils supporting the wall, MSE backfill, and retained backfill; and
- surcharges.

Wall Supplier

The approved wall supplier will check the external stability with respect to sliding, overturning, and bearing pressure. The Department will determine the need for any changes indicated by the wall supplier's external stability analysis.

The wall supplier is responsible for the internal stability design and for all costs associated with modifications to the overall wall geometry due to internal stability design or construction convenience.

21.2.2. External Stability and Internal Stability

The external stability calculation should include a check for sliding, overturning, rotational failure, and bearing pressure. Establish the wall geometry (including the width of reinforcement and height) based on these items for each height of wall.

If the wall supplier must increase the reinforcement width or height of the backfill due to internal stability requirements, the contractor is not paid for quantity increases.

The foundation engineer verifies increases over that required for external stability to ensure that the increase is justified.

21.2.3. Loads from Other Structures

Design MSE walls that support structures for the lateral and vertical loads imposed on the MSE wall. These loads can be substantial. The contract documents should identify the magnitude of the force and where the force is applied on the MSE wall in the "Footing Data Table." See Table 17.2-3.

21.2.4. Barrier Rails

MSE walls that incorporate a barrier rail at the top of the wall require special attention. The top of MSE walls are not strong enough to resist traffic impacts.

Design the wall to transfer traffic impacts from the barrier rail into a reinforced concrete slab that is located just below the roadway pavement. The concrete slab needs to be sufficiently massive to keep vehicular impact forces from being transferred into the MSE wall.

Size the concrete slab to resist sliding and overturning forces due to vehicular impacts, wind, or seismic loading as appropriate. Provide a minimum of 2 inches of separation between the bottom of the slab and the top layer of reinforcement.

21.2.5. Copings

Use CIP concrete copings at the top of all MSE walls. The top of the walls generally project 1 foot to 2 feet above the top layer of soil reinforcement. The coping holds together this unbraced section. Reinforcing steel from the top wall panels should extend into the coping.

21.2.6. Shop Drawings

The wall supplier prepares the shop drawings and supporting calculations. The bridge engineer reviews and approves the wall geometry and facing panel details. The calculation package is sent to the foundation engineer for review and approval.

21.3. Other Structures

21.3.1. Buried Structures

Use Section 12 of the *LRFD Specifications* for the structural design of buried structures or culverts. DOT&PF has developed standard designs that apply in most cases. See the DOT&PF *Standard Drawings*. A special design may be required when:

- the culvert geometry or height of soil above the culvert exceeds the values in the DOT&PF standard designs;
- loads are imposed on the culvert from other structures;
- the sequence of backfilling the sides of the culvert will not allow equal loading, or;
- special inlet, outlet, confluence, or other special hydraulic structures are needed for which a standard does not exist.

21.3.2. Signs, Signals, and Luminaire Structures

For these structures, including their foundation design, DOT&PF has adopted the AASHTO *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*

Observed problems with these structures include welded connections and anchor bolts. The AASHTO *LFRD Specifications* discuss the design and detailing of welded connections and anchor bolts. See the AASHTO/AWS *Structural Welding Code - Steel, D1.1* for additional discussion on welding issues.

DOT&PF has developed standard designs that apply in most cases. Occasionally, the Bridge Section will design structural supports for these roadside appurtenances. The Engineer-of-Record is also responsible for approving shop drawings on these structures.

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22. Reserved

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23. Bridge Rehabilitation

23.1. Introduction

Properly timed bridge maintenance and rehabilitation can maximize the service life of a bridge and delay the need for its replacement. This work will minimize the probability that bridges will deteriorate to an unsafe or unserviceable condition and protects the large capital investment in Alaska's inventory of bridges.

23.1.1. Scope of Work Definitions

Section 10.2.3 presents scope of work definitions to distinguish between the various levels of bridge work. Specifically, Section 10.2.3 presents definitions of the following:

- major bridge rehabilitation,
- minor bridge rehabilitation,
- bridge deck rehabilitation/replacement,
- seismic retrofit, and
- bridge widening.

Section 28.3 of this *Manual* discusses FHWA funding eligibility for bridge rehabilitation projects.

23.1.2. Rehabilitation Strategy

The development of a bridge rehabilitation project involves the following basic steps:

1. Collect the available data on the existing bridge (e.g., as-built plans, bridge inspection reports, load ratings, traffic volumes).

2. Perform a field investigation of the existing bridge. This may or may not be necessary.

3. Identify the necessary condition surveys and tests (e.g., coring, chain drag, chloride analysis, identifying fracture-critical members).

4. Evaluate the data from the condition surveys and tests.

5. Identify feasible rehabilitation strategies, estimate their costs, and compare these costs to the anticipated benefits.

6. Select and document the appropriate bridge rehabilitation strategy to upgrade the bridge to meet the necessary structural and functional objectives.

7. After the preliminary plans are completed, it is frequently a good idea to perform a field review to verify the design.

23.1.3. 3R Projects

Many bridge rehabilitation projects are identified as part of a highway 3R project.

For 3R projects, structural retrofits are required for any existing structural member with a capacity less than HS15 or HS20 for interstate bridges.

In addition to the 3R requirements in this *Manual*, follow the requirements in Section 1160.3.5 of the *Alaska Highway Preconstruction Manual*.

23.2. Documentation

23.2.1. Field Inspection

After reviewing relevant background material (e.g., as-built plans, shop drawings, bridge inspection reports, SI&A data, traffic data), the bridge engineer may determine that a field inspection is warranted. One objective is to identify the various condition tests and surveys that may be needed. Review the as-built plans before the field inspection.

During the field inspection note any areas of special concern (e.g., delamination, fatigue-critical details, bridge rail). Take photographs of approaches, elevation view, all four quadrants of the bridge, the feature being crossed, and any deficient features.

Ensure that all information is gathered as necessary to complete the Bridge 3R Memo.

In addition, verify that the bridge details match those shown in the as-built plans and shop drawings. Also, check for evidence of repair work or revisions not indicated in the plans and shop drawings.

Arrange for testing for the presence of lead-based paints and mill scale on steel structures.

23.2.2. Bridge 3R Memo

Figure 23-1 presents a sample bridge 3R memo, which should:

- document the findings from the field inspection, if conducted, including photographs;
- identify deficient items and provide recommendations for upgrade or repair;
- document the recommendations for seismic retrofit;
- make recommendations on the proposed bridge rehabilitation improvements;
- note scour susceptibility and provide a recommendation for upgrade or repair, if appropriate; and
- provide estimated design and construction project cost estimates.

Submit the bridge 3R memo to the project manager in the appropriate regional office.

23.2.3. Bridge Rehabilitation Literature

The design of new bridges is based primarily on the AASHTO *LRFD Bridge Design Specifications*. No single national publication exists that presents accepted practices, policies, and criteria for the rehabilitation of existing bridges as the *LRFD Specifications* provide for original design. However, the highway research community has devoted significant resources to identify practical, cost-effective methods to rehabilitate existing highway bridges.

Publications are readily available that may be of special interest when rehabilitating an existing bridge. Figure 23-2 provides a list of some of the more prominent publications that may be useful on a project-by-project basis.

MEMORANDUM

State of Alaska

Department of Transportation & Public Facilities
Statewide Design & Engineering Services Division/Bridge Section

TO:	Tiff Vincent Project Engineer Northern Region	**DATE:**	May 12, 2006
		FILE:	232, 233, 234, 235, 237, 238, 239
		TELEPHONE:	465-2975
		FAX:	465-6947
FROM:	Richard A. Pratt, P.E. Chief Bridge Engineer	**TEXT TELEPHONE:**	465-3652
CONTACT:	Drew Sielbach Bridge Management Engineer	**SUBJECT:**	AKSAS 61425 3R Evaluation, Chena Hot Springs Road MP 22-54

The estimated bridge 3R construction cost is $1,369,000. This includes 10% Mobilization and Demobilization costs, 15% Contingency, and 15% Construction Engineering. Recommended work includes:

North Fork Chena River, BN 232:
- Widen the pier cap.
- Install new bridge railing.
- Place backfill under both abutment caps.
- Install Name Place and Object Marker signs at both bridge ends.

North Fork Chena River, BN 233:
- Widen the pier cap.
- Install new bridge railing.
- Place backfill under one abutment cap.
- Install Name Place and Object Marker signs at both bridge ends.

North Fork Chena River, BN 234:
- Widen the pier caps.
- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

North Fork Chena River, BN 235:
- Widen the pier cap.
- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

"Providing for the movement of people and goods and the delivery of state services."

Figure 23-1 — Sample Bridge 3R Memo
Page 1 of 9

Tiff Vincent 2 May 11, 2006

North Fork Chena River, BN 237:
- Widen the pier caps.
- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

Angel Creek, BN 238:
- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

West Fork Chena River, BN 239:
- Widen the pier cap.
- Install new bridge railing.
- Place backfill under one abutment cap.
- Install Name Place and Object Marker signs at both bridge ends.

The 3R bridge analysis and cost estimates are attached.

Costs associated with seismic retrofit ($640,000.00) are eligible for funding using Federal Bridge Funds. Costs associated with all other work activities are not eligible for funding using Federal Bridge Funds.

The estimated Bridge PS&E development cost is $150,000. We believe it will take 6 to 8 months to develop the PS&E.

Figure 23-1 — Sample Bridge 3R Memo
Page 2 of 9

3R Bridge Analysis

BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	232	37.8	91.2	162.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following work items:

- Place backfill under both abutment caps to retain approach fill.
- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
205(4)	Porous Backfill	CY	20	$6,000
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	324	$64,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$13,800
	Contingencies		(15%)	$22,800
	Construction Engineering		(15%)	$22,800
TOTAL:				**$197,000**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	233	39.5	91.2	162.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:
- Place backfill under Steese abutment cap to retain approach fill.
- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
205(4)	Porous Backfill	CY	10	$3,000
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	324	$64,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$13,500
	Contingencies		(15%)	$22,300
	Construction Engineering		(15%)	$22,300
TOTAL:				**$192,700**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	234	44.0	90.4	182.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
401(1)	Class A Concrete	CY	12	$28,000
501(10)	Coring Concrete	LF	150	$36,000
502(1)	Post Tensioning (CIP Concrete)	Each	56	$19,600
503(1)	Reinforcing Steel	LB	3,600	$12,000
504(1)	Structural Steel	LB	2,000	$15,000
507(1)	Steel Bridge Rail	LF	324	$72,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$19,500
	Contingencies		(15%)	$32,200
	Construction Engineering		(15%)	$32,200
TOTAL:				**$278,800**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	235	45.7	90.4	122.0	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	244	$48,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$11,600
	Contingencies		(15%)	$19,100
	Construction Engineering		(15%)	$19,100
TOTAL:				**$165,400**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	237	48.9	90.4	182.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
401(1)	Class A Concrete	CY	12	$28,000
501(10)	Coring Concrete	LF	150	$36,000
502(1)	Post Tensioning (CIP Concrete)	Each	56	$19,600
503(1)	Reinforcing Steel	LB	3,600	$12,000
504(1)	Structural Steel	LB	2,000	$15,000
507(1)	Steel Bridge Rail	LF	364	$72,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$19,500
	Contingencies		(15%)	$32,200
	Construction Engineering		(15%)	$32,200
TOTAL:				**$278,800**

Figure 23-1 — Sample Bridge 3R Memo Page 7 of 9

BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
Angel Creek	238	49.9	91.2	82.0	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge satisfies the 3R seismic criteria. Therefore, no seismic retrofit is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
507(1)	Steel Bridge Rail	LF	164	$32,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$4,500
	Contingencies		(15%)	$7,400
	Construction Engineering		(15%)	$7,400
TOTAL:				**$63,600**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
West Fork Chena River	239	52.4	91.2	163.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Place backfill under Chena Hot Springs abutment cap to retain approach fill.
- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
205(4)	Porous Backfill	CY	10	$3,000
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	324	$64,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$13,500
	Contingencies		(15%)	$22,300
	Construction Engineering		(15%)	$22,300
TOTAL:				**$192,700**

Figure 23-1 — Sample Bridge 3R Memo
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1. *FHWA Workshop*, "Rehabilitation of Existing Bridges," 1984 (Revised 1986)
2. *NCHRP Report 206*, "Detection and Repair of Fatigue Damage in Welded Highway Bridges," 1978
3. *FHWA-RD-78-133*, "Extending the Service Life of Existing Bridges by Increasing Their Load-Carrying Capacity," 1978
4. *NCHRP Report 222*, "Bridges on Secondary Highways and Local Roads — Rehabilitation and Replacement," 1980
5. *NCHRP Report 226*, "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members," 1980
6. *NCHRP Project 12-17 Final Report*, "Evaluation of Repair Techniques for Damaged Steel Bridge Members: Phase I," 1981
7. *NCHRP Report 243*, "Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads," 1981
8. *NCHRP Report 244*, "Concrete Sealers for Protection of Bridge Structures," 1981
9. *FHWA-RD-82-041*, "Innovative Methods of Upgrading Deficient Through Truss Bridges," 1982
10. *FHWA-RD-83-007*, "Seismic Retrofitting Guidelines for Highway Bridges," 1983
11. *NCHRP Report 271*, "Guidelines for Evaluation and Repair of Damaged Steel Bridge Members," 1984
12. *NCHRP Report 280*, "Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members," 1985
13. *NCHRP Synthesis of Highway Practice 119*, "Prefabricated Bridge Elements and Systems," 1985
14. *NCHRP Report 293*, "Methods of Strengthening Existing Highway Bridges," 1987
15. *NCHRP Report 297*, "Evaluation of Bridge Deck Protective Strategies," 1987
16. *NCHRP Report 312*, "Condition Surveys of Concrete Bridge Components," 1988
17. *NCHRP Report 321*, "Welded Repair of Cracks in Steel Bridge Members," 1989
18. *NCHRP Report 655, "*Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements," 2010
19. *NCHRP Synthesis of Highway Practice 398*, "Cathodic Protection for Life Extension of Existing Reinforced Concrete Bridge Elements," 2009
20. *NCHRP Report 604*, "Heat-Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance," 2008
21. *Task Force 30 Report*, "Guide Specifications for Concrete Overlay of Pavements and Bridge Decks," 1990
22. *SHRP-S-344 "*Rapid Concrete Bridge Deck Protection, Repair and Rehabilitation," 1993
23. *SHRP-S-360*, "Concrete Bridge Protection, Repair and Rehabilitation — A Methods Application Manual," 1993
24. National Park Service "The Secretary of the Interior's Standards for Rehabilitation," 1992
25. National Park Service "The Secretary of the Interior's Standards for the Treatment of Historic Properties," 1992

Figure 23-2
Bridge Rehabilitation Literature

23.3. Bridge Condition Surveys and Tests

This section presents DOT&PF policies and practices for condition surveys and tests for a bridge rehabilitation project.

The discussion does *not* pertain to any condition surveys and tests performed for the Alaska Bridge Inspection Program (see Chapter 26) nor the DOT&PF Bridge Management System (see Chapter 28).

23.3.1. General

The bridge engineer is responsible for arranging and conducting field surveys, requesting specific tests performed by others (e.g., Statewide Materials), and evaluating data collected during the field survey and provided by others.

23.3.2. Concrete Bridge Decks

This section applies to deck-on-girder bridges. Concrete bridge decks include the structural continuum directly supporting the riding surface plus expansion joints, curbs, barriers, approach slabs, and utility hardware (if suspended from the deck). The bridge deck and its appurtenances:

- support and distribute wheel loads to the primary structural components;
- protect the structural components beneath the deck;
- provide a smooth riding surface; and
- provide a safe passageway for vehicular and bicycle/pedestrian traffic (e.g., skid-resistant surface, bridge rails, guardrail-to-bridge-rail transitions).

Any deterioration in these functions warrants investigation and possible remedial action.

The most common source of concrete bridge deck deterioration is the intrusion of chloride ions from roadway deicing agents into the concrete. The chloride causes formation of corrosive cells on the steel reinforcement, and the increased volume of the corrosion product (rust) induces stresses in the concrete resulting in cracking, delamination, and spalling.

Chloride ion (salt) penetration is a time-dependent phenomenon. There is no proven way to absolutely prevent penetration, but it can be slowed such that the service life of the deck is not less than that of the remaining structure.

Chloride penetration is, however, not the only cause of bridge deck deterioration. Other significant problems include:

- **Freeze-Thaw**. Results from inadequate air content of the concrete. Freezing of the free water in the concrete causes random, alligator cracking of the concrete and then complete disintegration. There is no known remedy other than replacement.
- **Impact Loading**. Results from vehicular kinetic energy released by vertical discontinuities in the riding surface, such as surface roughness, delamination, and inadequately set or damaged expansion joints. Remedial actions are surface grinding, overlay or replacement of deck concrete, and rebuilding expansion joints.
- **Abrasion**. Normally results from metallic objects, such as snow plow blades, chains, or studded tire wear. Remedial actions are surface grinding or overlay.

Certain factors are symptomatic indicators that a bridge deck may have a shorter than expected service life or that it is actually in the latter phases of its service life. Some examples are:

- extensive cracking (shrinkage, stress, etc.);
- extensive delamination;
- exposed reinforcing steel; and
- spalls.

The deck can be placed into one of the following categories (based on NBI ratings):

- Very good decks that need little attention. These are the (8) and (9) rated decks.
- Decks that are in reasonably good shape and need no substantial repair, but nominal maintenance expenditure can extend deck life. These are the (7) rated decks. Decks in this condition range most likely need some minor crack sealing and/or minor patching.
- Decks that need considerable repair but are still quite sound and capable of serving adequately for five to ten more years. These are candidates for repair and overlay with some type of non-permeable concrete. These are the (5) and (6)

rated decks. The bridge engineer would most likely consider an overlay for bridge decks in this condition range, depending on the extent of chloride contamination.

- Decks that are no longer serviceable and will soon need replacement regardless of any remedial action. Significant expenditures of funds are not justified until replacement. These are the (3) and (4) rated decks. Decks in these conditions fall into the “replace or rehabilitate deck” category, with depth of rehabilitation determined by the chloride content. Typically, the as-built concrete cover over the top mat of rebar is not well known and care must be taken in removing this concrete.

When considering a bridge for deck rehabilitation, the Bridge Section may request a number of tests to collect data on the deck condition. The data allows the bridge engineer to determine whether deck rehabilitation or deck replacement is appropriate and, if the choice is rehabilitation, the information allows the determination of the appropriate type of treatment.

A deck evaluation may include gathering the following information:

- a plot locating existing delaminations, spalls, and cracks;
- representative measurements of crack width;
- measurements of the depth of cover to the top mat of reinforcing steel;
- sampling and laboratory analysis to determine the existing levels of chloride contamination;
- measurements of electrical potential on a grid pattern to locate areas of active corrosion; and
- deck concrete compressive strength assessed through destructive testing of deck core samples.

Expect to obtain at least some degree of confirmation and conflicting test results because these field tests each have a degree of uncertainty. Thus, sampling multiple locations within a traffic lane is important to estimate the true condition of the deck and the extent of active corrosion.

Apply engineering judgment when analyzing multiple test results. The following provides more information on each type of data collected and their use in determining an appropriate deck treatment.

Visual Inspection

Description: A visual inspection of the bridge deck should establish:

- the approximate extent of cracking, representative crack widths, and spalling;
- evidence of any corrosion;
- evidence of pattern cracking, efflorescence, or dampness on the deck underside;
- rutting of the riding surface and/or ponding of water;
- performance of expansion joints;
- functionality of deck drainage system; and
- bridge rails and guardrail-to-bridge-rail transitions meeting current DOT&PF standards.

Purpose: Visual inspection of the bridge deck will accomplish the following:

- By establishing the approximate extent of cracking and crack width, corrosion, delamination, and spalling (and by having evidence of other deterioration), the bridge engineer can determine if a more extensive inspection is warranted.

- The inspection will identify substandard roadside safety appurtenances.

When to Use: On all potential deck rehabilitation projects.

Analysis of Data: Pattern cracking, heavy efflorescence, or dampness on the deck underside suggests that this portion of the deck is likely to be highly contaminated and active corrosion is taking place. In addition, the bridge engineer should consider:

- traffic control that will be required,
- timing of repair,
- age of structure,
- average annual daily traffic (AADT),
- slab depth,
- structure type, and
- depth of cover to reinforcement.

Delamination Sounding

Description: Establishes the presence of delamination, based on audible observation, by chain drag or hammer. Based on the observation that delaminated concrete responds with a “hollow sound” when struck by a metal object. See ASTM D4580 “Standard Practice for Measuring Delaminations in

Concrete Bridge Decks by Sounding." See Appendix 26.A for more discussion.

Purpose: To determine the location and area of delamination.

When to Use: On all concrete deck rehabilitation projects, except where asphalt overlays prevent performance of the test.

Analysis of Data: Decisions are made on a case-by-case basis, but the following recommendations can assist decision-making based on the extent of the bridge deck spalling:

- Consider remedial action when 10 percent of the surface area is delaminated.
- Consider bridge deck rehabilitation when 40 percent of the surface is delaminated.

Chloride Analysis

Description: A chemical analysis of pulverized samples of concrete extracted from the bridge deck. Determines concentration of water-soluble chlorides by using the *Gravimetric Method —Silver Chloride Method* as described in *Scott's Standard Methods of Chemical Analysis*, 6th Edition, March 1962, (D. Van Nostrand).

As an option, chloride testing may be conducted using ~~potentiometrictitaration~~ potentiometric titration with silver nitrate per AASHTO T 260 *Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials*. See Appendix 26.A for more discussion.

Purpose: To determine the chloride content profile from the deck surface to the top mat of rebar.

When to Use: Use on bridge decks where the need for major rehabilitation or replacement is anticipated. Take chloride samples at three to five locations along the travel lane per span from each span 100 feet or less in length. Increase the number of samples for longer spans.

Analysis of Data: The "threshold" or minimum level of water-soluble chloride contamination in concrete necessary to corrode reinforcing steel is approximately 255 ppm or 1.2 lbs of chloride per cubic yard in the concrete down to the top mat of rebar. Chloride concentrations of less than this threshold indicate a sound deck that will in most cases not require rehabilitation. Consider adding a deck protection system.

Chloride concentrations within or greater than this range above the top reinforcing mat require the removal of at least enough concrete so that the remaining concrete contamination is below the threshold.

Threshold or greater chloride concentrations at the level of the top reinforcing mat require either demolition removing enough concrete to ensure that the remaining concrete is below the threshold values, or possibly deck replacement. Threshold contamination or worse at or near the level of the bottom mat of reinforcing steel may require deck replacement.

Coring

Description: Take 2-inch or 4-inch diameter cylindrical cores. Avoid cutting into the rebar. In decks with large amounts of reinforcement, it is difficult to avoid cutting steel if 4-inch diameter cores are used.

Purpose: To establish strength, composition of concrete, crack depth, position of reinforcing steel.

When to Use: On all concrete deck rehabilitation projects when doubt exists on the compressive strength or soundness of the concrete or if the visual condition of the reinforcement is desired.

Analysis of Data: Less than 2 inches of concrete cover is inadequate for corrosion protection. If compressive strengths are less than 3 ksi, the bridge engineer must determine whether to proceed with the deck rehabilitation or to proceed with a deck replacement.

Pachometer Readings

Description: The pachometer produces a magnetic field in the bridge deck and displays a disruption in the magnetic field (e.g., induced by a steel reinforcing bar).

Purpose: To determine the location and depth of steel reinforcing bars. Establish these properties to a depth of approximately 4 inches.

When to Use: Use pachometer readings on all concrete rehabilitation projects to verify reinforcement location as needed. Use to locate steel to avoid damage when drilling or coring concrete.

Pull-Off Test

Description: The pull-off test determines the perpendicular tensile force that a surface area can resist before a plug of material is detached from the concrete deck per ASTM D7234 – 05 "Standard Test Method for Pull-Off Adhesion Strength of Coatings on Concrete Using Portable Pull-Off Adhesion Testers." Failure occurs along the weakest plane within the system comprised of the overlay and concrete deck. Clean the surface of the deck to obtain a dry deck.

Purpose: Determines the soundness of concrete to successfully receive an overlay.

When to Use: Use pull-off tests when an overlay is applied to a concrete deck of questionable soundness.

23.3.3. Superstructure

As defined in this *Manual*, the superstructure consists of the bearings and all of the components and elements resting upon them. The following briefly describes those condition surveys and tests that may be performed on the superstructure elements (other than bridge decks) to determine the appropriate level of rehabilitation.

Visual Inspection

Description: A visual inspection of the superstructure should include an investigation of the following to supplement the information contained in the NBI bridge inspection report:

- surface deterioration, cracking, and spalling of concrete
- major loss in concrete components
- evidence of efflorescence
- corrosion of reinforcing steel or prestressing tendons
- section loss in exposed reinforcing steel or prestressing tendons
- peeling coating system
- corrosion of structural metal components
- section loss in metal components due to corrosion
- cracks in metal components
- measurement of deformed shapes
- loose or missing rivets or bolts
- deterioration and loss in wood components
- collision damage by vehicles, vessels, or debris
- leaking expansion joints
- ponding of water on abutment seats
- measurement of deformed shapes, which can contribute to moment magnification
- condition and functionality of bearings
- distress in pedestals and bearing seats

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted. This is the primary method used to assess the condition of the superstructure.

When to Use: On all bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

Fracture-Critical Members (Steel)

Description: Fracture-critical members or member components (FCMs) are steel tension members or steel tension components of members whose failure would likely result in a partial or full collapse of the bridge. The Bridge Section has identified all fracture-critical structures in Alaska. The bridge engineer must recognize typical fracture-critical details when conducting the field review because it may affect the scope of bridge rehabilitation. Typical bridges in Alaska containing fracture-critical members are:

- steel trusses (pins, eye-bars, bottom chords, other tension members);
- two-girder steel bridges;
- transverse girders (supporting longitudinal beams and girders); and
- pin-and-hanger connections (located on suspended spans or at transverse girders).

Purpose: To identify FCMs in the project. Distress of FCMs requires special consideration.

When to Use: On all bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

Cracking in Steel

Description: If visual inspection by the bridge inspector reveals cracking in steel components, establish the extent and size of cracks to determine the appropriate remedial action. The following are the most common test methods performed by DOT&PF to locate cracks in steel components and measure their extent and size:

1. **Dye-Penetrant Testing (PT)**. This is the primary method used. Clean and paint the surface of the steel with a red dye. Allow time for the dye to "dwell" on the area and then wipe off. If a crack is present, the dye penetrates the

crack through capillary action. To indicate where the red dye "bleeds" from the crack, paint a white developer on the cleaned steel.

2. **Magnetic-Particle Testing (MT)**. Clean and sprinkle the surface of the steel with fine iron filings while a strong magnetic field is induced in the steel. A crack causes an interruption in the lines of magnetic flux, allowing them to "leak" from the metal, thereby attracting the metal filings, which form a trace along the line of the crack.

3. **Ultrasonic Testing (UT).** Testing devices that use high-frequency sound waves to detect cracks, discontinuities, and flaws in materials. The accuracy of UT depends upon the expertise of the individual conducting the test and interpreting the results.

4. **Eddy Current Testing (ET).** Eddy current testing uses the phenomenon of electromagnetic induction to detect flaws in conductive materials. This form of testing detects flux leakage emanating from a discontinuity in metal when an eddy current passes through the material. Eddy current testing can detect very small cracks in or near the surface of the material, the surfaces need minimal preparation, and physically complex geometries can be quickly investigated.

5. **Radiographic Testing (RT).** Radiographic testing uses X-rays, produced by an X-ray tube, or gamma rays, produced by a radioactive isotope. The basic principle of radiographic inspection of welds is the same as that for medical radiography. Penetrating radiation is passed through a solid object, in this case a weld rather that part of the human body, onto a photographic film, resulting in an image of the object's internal structure being deposited on the film. The amount of energy absorbed by the object depends on its thickness and density. Energy not absorbed by the object will cause exposure of the radiographic film. These areas will be dark when the film is developed. Areas of the film exposed to less energy remain lighter. Therefore, areas of the object where the thickness has been changed by discontinuities, such as porosity or cracks, will appear as dark outlines on the film. Inclusions of low density, such as slag, will appear as dark areas on the film while inclusions of high density, such as tungsten, will appear as light areas. All discontinuities are detected by viewing shape and variation in density of the processed film.

At a minimum, a Level II ASNT certified technician must conduct all tests. For more information, see *Detection and Repair of Fatigue Damage in Welded Highway Bridges*, NCHRP Report 206, July 1979.

Purpose: To quantify the extent of fatigue cracking,

When to Use: In rehabilitation projects where fatigue cracks are identified.

Analysis of Data: As required for each specific testing technique.

Load-Induced Fatigue Analysis (Steel)

Description: Fatigue is defined as steady-state crack growth. Failure of the component can result from growth of existing flaws in steel members to a critical size at which fracture is no longer effectively resisted by the toughness of the steel. The crack growth is a function of:

- crack size;
- location of crack (i.e., stress concentration at the structural detail);
- toughness (energy-absorbing characteristics of metal);
- temperature; and
- frequency and level of nominal stress range (transient stresses).

Purpose: To establish type and urgency of remedial action.

When to Use: Where cracks, found by visual inspection, are believed to be caused by fatigue or at fatigue-prone details.

Analysis of Data: A structural engineer experienced in fatigue-life assessment should perform the analysis. Establish fatigue characteristics of the metal for the analysis. For the stress range, the *LRFD Specifications* provide an upper-bound criterion of 75 percent weight of one design truck plus impact per bridge. The actual stress range of a given bridge component may be far lower than that specified by the *LRFD Specifications*, and it may be warranted to establish it by physical means.

Load Testing (Steel and Concrete)

Description: Measuring strains in girders can capture the actual live-load distribution. In most cases, these measured strains suggest an enhanced distribution compared to the empirical values in the applicable AASHTO bridge specifications.

Purpose: To determine more favorable load ratings.

When to Use: Where unacceptable load ratings result from the empirical distribution equations. The DOT&PF bridge instrumentation program can be used to quantify the live-load distribution for use in load ratings.

Sounding and Penetration Tests (Timber Bridges)
Description: Sounding tests using a hammer and/or penetration tests using an ice pick. See Section 26.5.5, Special Bridge Inspection Practices, and Appendix 26.A for optional equipment and tests.

Purpose: To establish the soundness of wood components.

When to Use: Where the soundness of wood components is in question.

23.3.4. Substructures and Foundations

Visual Inspection
Description: A visual inspection of the substructure components should address the following to supplement the NBI bridge inspection report:

- surface deterioration, cracking, and spalling of concrete
- major section loss in concrete components
- evidence of corrosion in reinforcing steel
- section loss in exposed reinforcing steel
- deterioration or loss of integrity in wood components
- leaking joints and cracks
- deformations contributing to moment magnification
- collision damage
- changes in geometry such as settlement, rotation of wingwalls, tilt of retaining walls, etc.
- seismic vulnerabilities
- accumulation of debris
- erosion of protective covers
- changes in embankment and water channel
- evidence of significant scour

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

When to Use: On all potential bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

Delamination Sounding
The relevant superstructure criteria apply to substructures. See Section 23.3.2.

Soundings and Penetration Tests (Timber Bridges)
The relevant superstructure criteria apply to substructures. See Section 23.3.3.

FCM Pier Caps with Steel
The relevant superstructure criteria apply to substructures. See Section 23.3.3.

23.3.5. Special Provisions

If a special provision is required for a bridge rehabilitation project, address some of the key issues including:

- surface preparation,
- materials specifications,
- types of equipment,
- bonding methods/specifications,
- curing,
- protection of adjacent bridge elements,
- repair of damage,
- weather limitations,
- required testing after construction, and
- opening to traffic.

23.4. Bridge Deck Rehabilitation

23.4.1. General

Chapter 16 provides an in-depth discussion on the design of bridge decks that are constructed compositely in conjunction with concrete and steel girders for new bridges. Many of the Chapter 16 design and detailing practices may also apply to deck rehabilitation.

23.4.2. Patching

A permanent repair can be assured only if all concrete is removed in areas having a chloride content sufficient to sustain corrosion.

For partial depth repairs, remove concrete to a depth of ¼ inch plus the maximum size of the aggregate below the bottom of the top mat of reinforcing steel. The actual corrosion threshold can be as low as 1.3 lb of Cl per cubic yard of a typical deck concrete, but a value of 2 lb of Cl per cubic yard is commonly accepted as the level beyond which removal of the concrete is warranted. Unless the contaminated concrete is removed, differences in the surface conditions on the reinforcing bar may cause the formation of anodic and cathodic areas and a resumption of the corrosion process. However, removal of concrete below the reinforcing steel may be extremely costly, and complete removal and replacement of the deck may be more economical.

An evaluation of the corrosion process indicates that patches cannot be considered permanent repairs, and field experience tends to verify this conclusion. Newly delaminated areas are often found adjacent to areas patched months before. Nevertheless, patching can be an appropriate temporary action until more extensive restoration is performed, and it can provide substantial service with the subsequent installation of a protective overlay.

A wide variety of materials have been used for patching bridge decks. Although conventional Portland cement concrete is often used, many other materials are available to provide rapid strength development and to allow early opening of the deck to traffic. It is essential to follow the manufacturer's requirements for mixing, placing, and curing. A polymer concrete overlay, if used, can also serve as the deck patching material.

Bonding components vary with the repair materials. Some prepackaged polymer-modified concretes develop sufficient adherence so that a bonding agent is not required. Consult the manufacturers of all prepackaged, fast-setting patching materials for the proper bonding agents. A methacrylate primer is used for polymer overlay patches.

23.4.3. Crack Repair

Epoxy-Resin Injection

Epoxy-resin injection is commonly used to fill cracks in decks. Because the resin is injected under pressure, it is usually possible to fill the entire depth of crack.

Methacrylate Sealant

A low-viscosity organic liquid compound is flooded over the deck, and fills the cracks by gravity and capillary action. Accordingly, the success of this operation depends on the crack size, selection of the appropriate compound, temperature, contamination on the crack walls, and the skill of the operator. The contractor must clean the deck surface prior to application of the sealant.

23.4.4. Waterproof Membrane/Asphalt Overlay

DOT&PF typically uses waterproof membranes with an asphalt overlay on new decked bulb-tee girder bridges, and the Department has experienced good results with their use. On deck-on-girder bridges, a waterproof membrane with asphalt overlay can also demonstrate better performance than concrete or polymer overlays for deck rehabilitation.

A waterproof membrane with asphalt overlay has comparable construction time frames as the other overlay systems. The surface preparation for the membrane is minimal. Only high points or exposed rocks must be removed so that they will not puncture the membrane.

Review bridges on highways programmed for pavement overlays to ensure safety, load capacity, and performance are not adversely affected. Consider the factors below in determining an acceptable overlay strategy.

Overlay Impact on Live Load Capacity

Review plans to determine what wearing surface dead load (asphalt thickness) the bridge was designed to accommodate.

Verify that the current load rating uses the proposed asphalt thickness. If not, recalculate load rating with proposed asphalt thickness.

On the NHS, inventory load ratings should not decrease below HS 25. Off the NHS, inventory load ratings should not decrease below HS20. If necessary, limit asphalt thickness to achieve these values.

Verify that the proposed asphalt thickness does not decrease the operating load rating for the route on which the bridge is located. If necessary, limit the asphalt thickness so that the route's operating rating does not decrease.

Overlay Impact on Bridge Rail Height

Verify that the proposed asphalt thickness does not decrease the rail height below the following:

1. For crash tested bridge railing, 1-inch plus or minus the crash tested height.
2. For non-crash tested bridge railing, 27 inch minimum height.

Overlay Impact on Bridge Deck Joints

If bridge deck joints exist, project options are:

1. Match existing expansion joint height. In some situations this may require tapering the asphalt thickness.
2. Adjust the expansion joint height to match the proposed asphalt thickness.

Need for a Waterproof Membrane

Place a waterproof membrane below asphalt overlays.

When the existing asphalt thickness allows the top wearing surface to be milled, review bridge inspection reports to determine if the deck is currently watertight.

If there is evidence of water actively leaking through the bridge deck, consider removing all asphalt and installing a new waterproof membrane.

23.4.5. Microsilica Concrete Overlay

Microsilica concrete overlays provide a durable, smooth and economical riding surface that is resistant to chloride penetration and delamination.

Microsilica is a pozzolanic material that is much finer than cement particles, which allows it to produce a denser matrix. One of the biggest advantages of a microsilica concrete overlay is its reduced permeability to chloride penetration. Compressive strength is enhanced as well.

Although microsilica concrete is highly resistant to chloride penetration, the permeability of the microsilica concrete is also highly dependent on the quality of construction and proper curing of the deck.

23.4.6. Polyester Concrete Overlay

Polyester concrete is a composite of dry aggregate in an unsaturated, or thermoset, polyester resin binder. When the liquid resin cures into a hardened, cross-linked state, this forms a polyester concrete. The primary features and resulting benefits are:

- ease of application for reduced production costs,
- quicker cure for shorter lane closure times,
- thinner overlays for greater live load capacity,
- higher elongation and tensile strength for improved dynamic performance,
- a protective barrier against moisture and deicers for lower maintenance costs and longer service life, and
- greater resistance to abrasion and impact for lower maintenance costs and longer service life.

23.4.7. Cathodic Protection

The advantage of cathodic protection is that it can halt the progress of corrosion without the removal of chloride-contaminated concrete. Corrosion requires an anode, a point on the reinforcing steel where ions are released.

Cathodic protection can be either passive or active. Passive protection uses an anode that is more electrochemically active than the reinforcing steel. Active protection uses an anode plus an impressed current to increase the current flow. Anodes vary in shapes, sizes and metals.

Cathodic protection is seldom used because of several disadvantages, including:

- need for expertise in design and construction,
- need for periodic adjustment, and
- power requirement.

For more information, see *Guide Specifications for Cathodic Protection of Concrete Bridge Decks*, 1994, AASHTO Task Force 29.

23.4.8. Joint Rehabilitation and Replacement

Joint rehabilitation refers to the repair of a portion of an existing joint and not complete replacement. Joint rehabilitation includes repairing or replacing loose or broken restrainers on strip seal expansion joints, failed header materials adjacent to joints or torn seals. In most cases, the failure is due to vehicle impact. Failure may also be due to incompressible materials in the joint.

Bridges with asphalt overlays require a concrete header adjacent to the expansion joint. Concrete headers should be at least 8 inches but preferably 12 inches wide. Deck concrete should be removed down to a distance below the top mat of reinforcing steel to provide development length for the new header reinforcement.

Use a minimum number of joint splices with a full-length seal preferred. Torn strip seals can be repaired by vulcanizing or gluing; however, vulcanizing is preferred.

Where joint rehabilitation is not feasible, a replacement of an existing damaged or malfunctioning joint may be necessary. Chapter 19 provides guidance on joint selection and design.

23.4.9. Upgrade/Retrofit Bridge Rails/Approach Rails

Evaluation

Section 16.5.1 presents DOT&PF practices for new bridge rails. Ideally, existing bridge rails on a bridge rehabilitation project will meet the criteria in Section 16.5.1 or will be replaced with a new, NCHRP-350 compliant bridge rail. However, this is not always practical for a variety of reasons (e.g., dead load considerations, incompatibility with an existing bridge deck).

Table 23-1 presents the requirements for addressing bridge rails on existing bridges. This table applies to the maintenance and rehabilitation of existing bridges, including existing bridges within the limits of 3R roadway projects.

In addition to Table 23-1, examine the following when evaluating an existing bridge rail:

1. **Critical Design Details**. Inspect the existing bridge rail to verify the integrity of critical design details, such as:

   a. base plate connections,
   b. anchor bolts,
   c. welding details,
   d. concrete cracking, and
   e. reinforcement development.

2. **Safety Deficiencies**. Review the maintenance and repair history of the bridge rail. Even with no history of impact damage, an inspection of the existing bridge rail may reveal inherent safety deficiencies in the rail design, such as:

   a. potential for snagging (i.e., no blockouts)
   b. presence of curb in front of bridge rail
   c. inadequate height
   d. inadequate guardrail-to-bridge-rail transition

23.4.10. Bridge Deck Strengthening

A deck is strengthened by either reinforcing the decking material or decreasing the distance between the deck supports. Strengthening a concrete deck involves adding a system of floorbeams and stringers between the existing longitudinal beams.

Strengthening any deck involves the addition of weight, which reduces the live-load capacity of the supporting members. A structurally deficient deck due to insufficient steel requires strengthening. Use structural analysis to determine the size and spacing of supports to provide the required capacity. The concrete floor slab must be capable of resisting the negative moments induced at the new support locations.

23.4.11. Deck Replacement

Precast concrete slabs are prefabricated as traditionally reinforced, prestressed, or a combination of the two. Panels are placed on the tops of the beams in a mortar bed with space or opening for shear connectors.

Cast-in-place (CIP) concrete slabs are cast on the tops of the beams encompassing the existing rehabilitated shear connectors or new replacement shear connectors.

Traffic control is a critical issue when considering a deck replacement.

TYPE OF PROJECT	BRIDGE RAIL ACTION
Preventive Maintenance (PM)	Replacement, retrofit, or repair of existing rail is not required, except when damaged beyond repair. When damaged beyond repair replace rail in accordance with Figure 16-10.* Consider in-kind repair to address obvious deficiencies. * Replacement may be deferred to a later project
3R with Major Bridge Rehabilitation or Widening (Widening on one or both sides)	Provide railing in accordance with Figure 16-10.
3R with Minor Bridge Rehabilitation (No widening and no work affecting the existing bridge rail)	Replacement, retrofit, or repair of existing rail is recommended, but not required, except if damaged beyond repair or does not meet height requirements, then replace in accordance with Figure 16-10. The height requirements are: 27” for low speed roadways (45 mph and less) 32” for high speed roadways (greater than 45 mph)* * 1 inch less tolerance allowed

Table 23-1
Bridge Rail Upgrade Procedures

23.5. Concrete Superstructures

Chapter 14 provides a detailed discussion on the design of concrete superstructures. Many of the Chapter 14 design and detailing practices also apply to the rehabilitation of an existing concrete bridge.

23.5.1. Remove/Replace Deteriorated Concrete

A clean, sound surface is required for any repair operation; therefore, all physically unsound concrete, including all delaminations, should be removed.

Verify that the concrete not removed is capable of resisting its weight, any superimposed dead load, live load (if the bridge will be repaired under traffic), formwork, equipment, and the plastic concrete without the need for supplemental temporary support. The formwork should resist the plastic concrete without slipping or bulging.

23.5.2. Crack Repair

Attempt to identify and remediate the mechanism causing the concrete to crack. Section 23.4.3 discusses crack repair for bridge decks. Use similar techniques for concrete superstructures.

23.5.3. Grouting

Because of the availability of epoxy injection, do not use grouting in crack repair unless the crack width is greater than 10 mm. Limit its application to filling post-tensioning ducts and to provide mortar-beds for precast concrete deck components, barriers, and bearings.

23.5.4. Post-Tensioning Tendons

The addition of post-tensioned tendons can restore the strength of prestressed concrete girders where original strands or tendons have been damaged. Strengthening by post-tensioning may also be applied to non-prestressed concrete girders.

Collision of over height vehicles or equipment with a bridge constructed with prestressed concrete girders may result in damage to or severing of the girder tendons. Exposure to water and salt may also cause damage, particularly where the concrete cover is damaged or cracked. Because the steel tendons determine the load-carrying capacity of the girder, any damage impairs resistance and must be repaired. The bridge engineer can use external longitudinal post-tensioning along the sides of pier caps to close transverse cracks and improve seismic performance.

At a minimum, the following steps apply:

- Conduct an investigation on the extent of damage.
- Perform a structural evaluation to determine the extent of repair.
- Evaluate the existing diaphragms to ensure their adequacy to support the end anchorage of the tendons.
- Determine the placement of the temporary load to be applied to the bridge prior to removal and placement of concrete in prestressed concrete girders, if any, to ensure the proper distribution of loads in the final condition.

Design the post-tensioning system in accordance with the manufacturer's recommendations. Wedge-type anchorages are susceptible to high seating losses for short-length tendons. High-strength prestressing bars are preferred in this application.

23.5.5. Bearings

Often, the existing bearings may only need cleaning or repositioning. Extensive deterioration or frozen bearings may indicate that the design should be modified. The bridge engineer may substitute a variety of elastomeric devices for sliding and roller bearing assemblies. If the reason for deterioration is a leak in the expansion joint seal, the seal should be repaired or replaced.

If the bearing is seriously dislocated, its anchor bolts badly bent or broken, or the concrete seat or pedestal is structurally cracked, the bridge may have a system-wide problem usually caused by temperature or settlement that warrants investigation.

See Chapter 19 for more information on bearings.

23.5.6. Design for Damaged Prestressed Concrete Beams

Definitions of High Load Damage[1]

Surface Damage: These are surface scrapes and small nicks less than one inch deep. This type of damage does not generally warrant repairs unless it is associated with other bridge maintenance repairs.

Minor Damage: Isolated concrete cracks, nicks, and spalls up to one inch deep with no reinforcing or

[1] Waheed, Kowal, Loo, Page2

prestressing strands exposed. Minor damage adversely affects the aesthetics; however, the structural capacity is not reduced. It is important to restore concrete cover to prevent reinforcing steel from eventually becoming exposed and corroded.

Moderate Damage: Concrete cracks and wide spalls exposing reinforcing steel and prestressing strand. There is no immediate effect on the structural capacity, although cracks and exposed reinforcement can reduce structure life due to corrosion and freeze thaw action.

Severe Damage: This includes damaged prestressing strands and reinforcing steel, significant loss of cross section, and possible lateral misalignment due to girder distortion.

Cracks[2]

Use the following definitions for crack width:

- Hairline – less than 0.01 inch
- Narrow – less than 1/64th inch and greater than or equal to 0.01 inch
- Medium – less than 1/32nd inch and greater than or equal to 1/64th inch
- Wide – greater than or equal to 1/32nd inch

Repair/Replacement Criteria

Surface Damage. Surface damage does not, in general, need to be repaired. If work is being done on the bridge, surface damage repair can be included in the other work if desired.

Minor Damage. Repair minor damage generally by epoxy injection. Cracks greater than 0.010 inch in width should be pressure injected full depth with an approved epoxy resin. Prestressed girders should be in compression in their unloaded state; no cracks should be visible in the unloaded state, thus all cracks indicate some amount of damage.

Moderate Damage. Moderate damage may be repaired without preloading the girder or tensioning any strands; however, preload is recommended[3].

Severe Damage. Severe damage requires repair or replacement of the girder.

[2] Waheed, Kowal, Loo, Page 5

[3] Shanafelt, Horn, Page A-52

Repair Criteria

For a precast, prestressed girder to be repaired, it must meet the following criteria:

- Less than 25 percent of the total number of prestressed strands damaged unless exterior post-tensioning is planned.[4]
- No abrupt lateral offsets have occurred in the girder due to impact damage.[5]
- The shear capacity of the proposed repaired section is greater than the demand anticipated.

Prestressed Tendon Repair

The following repair procedure does not apply to 0.6-inch strand. Should a damaged 0.6-inch strand require repair, new methods, procedures, equipment, and supplies may be required.[6]

Remove unsound concrete from around the damaged area, enough to expose sufficient undamaged portions of damaged strands to allow splicing operations as per manufacturer's recommendations. Note that many manufacturers recommend staggering splices if multiple strands are broken; thus, if the manufacturer recommends, it may be necessary to remove additional sound concrete to provide enough exposed strand to install a staggered splice. If a previously damaged strand is damaged again and the previous splices exposed by the new damage, cut out the old splice, cut the strand back far enough to splice to undamaged strand. If a new piece of strand must be spliced in, the couplers must be at least 3 feet apart as measured from the end of the couplers closest to each other.[7]

Install couplers/repair strands in such a way as to retension the strand to 60 percent of the specified ultimate strand strength.[8] For 0.5-inch, 270 ksi strand, this would be 162 ksi or 24.79 kips. See the manufacturer's instructions for methods to achieve this force.

Stressing methods must be submitted to the bridge engineer for approval.

[4] Shanafelt, Horn, Page 29

[5] Shanafelt, Horn, Page A-21 through A-22

[6] Banse, October 3, Harton, October 9

[7] Waheed, Kowal, Loo, Page 21

[8] Waheed, Kowal, Loo, Page 9

It is very important to check that all threaded components are free from defects and protect them from damage.

All threaded components should be lubricated prior to use.[9]

After stressing the strand, preload the girder, then unload the girder, and check the stress in the strand. If the strand is still tensioned as required, preload the girder and patch the concrete.

If the strand has lost more than 1 kip pretension, repeat the cycle of preload and retension until the loss is less than 1 kip. This is the slippage due to anchorage set. Once the anchorages have been set, place the girder and patch the concrete.[10]

Post-tensioned Tendon Repair

Use special care when repairing post-tensioned tendons. According to the main supplier the DOT&PF has historically used, post-tensioned tendons cannot be repaired without special equipment. Follow the manufacturer's recommendations for post-tensioned tendon repair.[11]

See Appendix 23A for the DOT&PF procedure for the construction repair of damaged prestressed concrete beams and a list of the cited references.

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[9] Zobel, Jirsa, Page 82, Harton, October 9

[10] Zobel, Jirsa, Page 81

[11] Harton, October 9

23.6. Steel Superstructures

Chapter 15 provides a detailed discussion on the structural design of steel superstructures for new bridges. Many of the Chapter 15 design and detailing practices also apply to the rehabilitation of an existing steel superstructure.

23.6.1. Fatigue-Crack Retrofit

Fatigue damage entails the formation of cracks in base metal or welds. If not repaired in a timely manner, fatigue cracks can lead to brittle fractures. The type of repair and its timing depend on many factors including:

- reason for the cracking (e.g., poor detailing, heavier than anticipated truck traffic, poor notch-toughness, load induced or distortion induced, constraint);
- location of the crack (e.g., cross frame, stiffener, weld, heat-affected zone, main member);
- depth, length, and geometry of the crack; and
- redundancy.

23.6.2. Fatigue-Cracking Countermeasures

Hammer Peening

Peening is an inelastic reshaping of the steel at the surface location of cracks, or of potential cracks, by using a mechanical hammer. This procedure not only smoothes and shapes the transition between weld and parent metal, it also introduces compressive residual stresses that inhibit the cracking. Peening is most commonly used at the ends of cover plates to reduce fatigue potential. However, the success of hammer peening is highly dependent upon the skill of the operator.

Ultrasonic Impact Treatment (UIT)

A computer-controlled peening process using high-speed peening called ultrasonic peening is available. This removes the dependency of the quality of mechanical-hammer peening on the operator's proficiency. This process promises weld-toe enhancement for unavoidable poor fatigue resistance details such as terminations of longitudinal stiffeners. It involves the deformation of the weld toe by a mechanical hammering at a frequency of around 200 Hz superimposed by ultrasonic treatment at a frequency of 27 kHz. The objective of the treatment is to introduce beneficial compressive residual stresses at the weld toe by plastic deformation of the surface and to reduce stress concentration by smoothing the weld toe profile.

For more information, see "Fatigue Resistance of Welded Details Enhanced by Ultrasonic Impact Treatment (UIT)" by Sougata Roy, John W Fisher, Ben T Yen in the International Journal of Fatigue, Volume 25, Issues 9–11, September–November 2003, Pages 1239–1247.

23.6.3. Section Losses

The following options are available to correct section losses by adding doubler plates:

1. **Welding Doubler Plates**. It is common practice to use welding for shop fabrication of steel members and for welding pieces in preparation for rehabilitation work. Field welding is often difficult to perform properly in high-stress areas, and this requires individuals with the necessary skill and physical ability. The proper inspection of field welds is equally difficult. Preferably, use a shop weld instead of a field weld.

   Field welding should only be allowed on secondary members, for temporary repairs, or in areas where analysis shows minimal fatigue stress potentials.

2. **Bolting Doubler Plates**. Bolting doubler plates over a corroded section (that has been cleaned and painted to prevent further section loss) is a more reliable long-term solution.

23.6.4. Add Cover Plates

If the deck is deteriorated and removed, adding cover plates to strengthen a beam may be a viable strategy. However, the *LRFD Specifications* place the ends of fully welded cover plates into Fatigue Category E or E'. The advantages of adding cover plates may be offset by introducing fatigue-prone details. If bolts designed in accordance with LRFD Article 6.10.12.2.3 are used at the end of the cover plates, apply Fatigue Category B. Because this requires the presence of drilling equipment and work platforms, consider a fully bolted cover plate construction.

23.6.5. Introduce Composite Action

Introducing composite action between the deck and the supporting beams is a cost effective way to increase the strength of the superstructure. The *LRFD Specifications* mandate the use of composite action where current technology permits. Welded studs or high-strength bolts can achieve composite action. Design shear connectors in accordance with LRFD Article 6.10.10.

Composite action considerably improves the strength of the upper flange in positive moment areas, but its beneficial effect on the beam as a whole is only marginal. The combination of composite action in conjunction with selective cover plating of the lower flange is the most effective way of beam strengthening.

Introducing composite action near joints prevents the deck from separating from the beams, thus increasing the service life of the deck. Provide this on each bridge that will have its deck removed for other reasons.

23.6.6. Add New Girders

If the deck is removed, a new set of girders added to the existing bridge is one alternative to strengthen the superstructure. To ensure proper distribution of live load, rigidity of the new girders should be close to that of the existing ones.

23.6.7. Bearings

The discussion in Section 23.5.5 also applies to steel superstructures.

23.6.8. Painting

Technically, bridge painting is maintenance work and not rehabilitation work, but frequent painting is considered in conjunction with rehabilitation work on steel structures.

Painting is generally warranted for bridges experiencing severe corrosion with section loss. It should also be considered for highly visible bridges with rust Grade 4 or more (SSPC Vis 2) where 10 percent of the surface area is rusted. This level of corrosion is aesthetically unacceptable and will progress to steel section loss.

Much of the interior of Alaska has a benign climate with slow corrosion rates where zone (spot repair) painting at expansion joints may be appropriate. Complete superstructure re-painting may be more appropriate for bridges located in coastal areas having a more aggressive climate with accelerated corrosion rates. Consider the following three options:

1. full removal of existing paint and repainting,
2. a complete recoat over the top of the existing paint (overcoat), or
3. zone (spot repair) painting.

An important factor is that virtually all paint applied to bridges prior to 1988 contains lead and other heavy metals. To remove existing paint, the current state of practice is abrasive blast removal, full enclosure, with environmental and worker monitoring.

The paint industry has developed products that can be successfully applied over existing paints. An overcoat may be an economic alternative to full removal and repainting where a uniform appearance is desired at the conclusion of the project; however, the removal of the lead-based paint is deferred until a subsequent rehabilitation or structure replacement. Zone painting neither provides a uniform appearance nor removes the lead-based paint. Zone painting may be appropriate in localized areas where corrosion could cause section loss.

Consider the proper selection of paint for an overcoat. An improperly specified or improperly applied overcoat can cause failure of the original paint that was performing satisfactorily. Review the manufacturer’s literature on any paint’s service environment and recommended application environment.

Overcoating lead-based paint having extensive rust spots but less than 1 percent of surface rusted (rust Grade 6) is generally not cost effective. The existing top coat is usually aluminum-based and too stiff and brittle to overcoat.

Blast cleaning must remove rust spots to bare metal and remove the aluminum top coat. Full containment is required.

Overcoating retains the existing lead-based paint and removal is merely deferred. It is generally more practical to postpone painting until the bridge has reached rust Grade 4 (10 percent of surface rusted) when complete lead-based paint removal and repainting is appropriate.

Overcoating existing zinc-based paint for rust Grade 6 will extend the service life of the existing coating. This option is especially attractive on bridges that are highly visible to the public.

DOT&PF typically specifies a single-component, moisture-cure, polyurethane paint system meeting applicable ASTM specifications. The specific paint must allow application in cold and damp conditions.

23.6.9. Heat Straightening

This technique is restricted to hot-rolled steels. Steels deriving their strength from cold drawing or rolling tend to weaken when heated. The basic idea of heat straightening is that the steel, when heated to an appropriate temperature, loses some of its elasticity and deforms plastically. This process rids the steel of built-up stresses.

Heat straightening is as much an art as science. To avoid overheating the steel, this technique should only be performed by those with experience in the process.

Heat straightening temporarily reduces the resistance of the structure. Apply measures such as vehicular restriction, temporary support, temporary post-tensioning, etc., as appropriate while the work is in progress.

For additional guidance on heat straightening, see *Guide for Heat-Straightening of Damaged Steel Bridge Members*, FHWA, 2008, and *Heat-Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance*, NCHRP, 2008.

23.6.10. Pin/Hanger Rehabilitation

Bridge engineers originally used pin and hanger details to facilitate the analysis of bridges by providing pins in otherwise continuous bridges. Today their use is not necessary due to modern computer-based structural analysis.

These details are particularly susceptible to corrosion. Corrosion can result in the initiation of fatigue cracking in the hangers due to frozen pins and the unseating of the hangers on the pins due to misalignment from the corrosion product.

The infamous collapse of one span of the Mianus River Bridge on I-95 in Connecticut was the result of corrosion of a pin and hanger detail.

Three solutions are possible for pin and hanger details:

1. **Unlock Frozen Pins and Hangers.** The pin and hanger detail can be disassembled after providing alternative support to the suspended girder. Then, the various components of the detail can be cleaned of rust and dirt or replaced before re-assembly.

2. **Provide a Catch Girder.** As a safeguard against failure, especially for fracture-critical girders, an alternative permanent support system can be fabricated to "catch" the suspended girder ends if the pin and hanger detail fails. Such a structure must be temporarily provided to perform the unlocking of frozen details discussed above.

3. **Eliminate the Pin and Hanger Detail.** If the girder sections allow, a bolted splice of the web and flanges can be fabricated to replace the pin and hanger. A structural analysis of the resulting continuous structure must verify that the resulting loads do not exceed the resistance of the existing girder section.

23.6.11. Post-Tensioning

External post-tensioning can be applied to both steel and concrete beams to reduce tensile stresses, to strengthen beams, or to make simply supported beams continuous. There are a variety of successful methods of post-tensioning in the literature.

The *LRFD Specifications* require the establishment of resistance at ultimate limit states at which the interaction between the parent and the post-tensioning systems should be investigated.

Because they are always close to the beam ends, post-tensioning anchorages are vulnerable to salt-laden water seeping through imperfectly sealed deck joints. Protect the tendons by corrosion-resistant ducts, either grease filled or grouted, especially if being exposed to airborne salts such as at overpasses.

23.7. Substructures/Foundations

Chapter 18 provides a detailed discussion on the structural design of substructures for new bridges, and Chapter 17 applies to foundations. Many of the Chapter 17 and Chapter 18 design and detailing practices also apply to the rehabilitation of the substructures of an existing bridge.

23.7.1. Deadman Anchorages

The lateral force exerted by retained earth tends to push forward and rotate abutments and retaining walls. One solution for this problem is the installation of a deadman anchor.

A deadman is a heavy solid mass, usually concrete blocks that are connected to the retaining structure by long steel rods. A deadman is located in a stable earth mass well behind the structure. For wingwalls, or walls located on both sides of the roadway, they can simply be connected together by steel tension rods.

Protect the rods against corrosion, and consider the effects of differential settlement.

Because this stabilization technique modifies the wall support from a cantilever to simple span pinned, check the wall reinforcement for the revised moments. The lateral earth pressure diagram may also be changed if more than one level of tension rods and anchors are installed.

23.7.2. Column Repair/Rehabilitation

The typical repair of concrete columns is to place a concrete jacket around the member to protect it from further deterioration or to restore its structural integrity.

The repair can be made with a standard wood or metal form work which is removed after curing or a fiberglass form that remains in place and helps protect the surface of the member.

In areas of moderate to high seismicity, consider installing a grouted steel jacket to both repair the column and provide enhanced seismic performance.

23.7.3. Post-Tensioning

Inadequately reinforced concrete pier caps may require strengthening by external post-tensioning.

Evaluate the existing concrete in the cap. Include tensioning strand or rods externally on the cap to add compression to the cap. Use brackets, distribution plates, and other components to transfer the post-tensioning forces to the cap. If aesthetics are a concern, widen the cap with ducts placed internally for the post-tensioning.

Post-tensioning is usually symmetrical to the cap so that an eccentric force is not introduced. Look at the stressing sequence to ensure that the cap is not overloaded eccentrically during post-tensioning operations.

23.7.4. Pile Section Loss Repair/Replacement

For steel piles, the following applies to section losses:

1. **Small Loss.** The restoration of the section of piles that experience a small loss of section associated with “normal” rusting is usually not warranted.

2. **Medium Loss.** When rusting has reduced the section of the pile such that it becomes a structural concern, the missing cross section is rebuilt by adding plates to the flanges and/or web as appropriate by either welding or bolting. Concrete cast in a stay-in-place fiberglass form can also be considered.

3. **Extensive Loss.** When the pile has deteriorated such that there is insufficient sound remaining material for the section to be rebuilt, use a new pile; the damaged pile may or may not be removed.

For wood piles, repair section losses by partial replacement, epoxy injection, jacketing, or some combination of these methods.

For information on wood piles, see “Timber Bridges – Design, Construction, Inspection and Maintenance” by M. A. Ritter, United States Department of Agriculture, Forest Service, EM 7700-8, June 1990, Chapter 14.

For concrete piers, section loss may be repaired by removing all deteriorated material, constructing a formwork for a jacket, placing a reinforcing steel cage of appropriate size in the formwork, and filling it with compacted concrete. The technique has extensive literature on its application.

23.7.5. Crutch Bent Repair

Rehabilitate settled or otherwise failing piers or abutments through a crutch bent. A crutch bent is a supplemental bent placed adjacent to a failing pier or abutment to restore support.

23.7.6. Tremie Concrete Encasement

When underwater footings or pile caps are undermined, one of the more common repair methods is to fill the void foundation area with a concrete. To place the concrete, use some type of formwork to confine it. A tremie encasement is a steel, wood, or concrete form that is placed around the existing footing or cap to reestablish the foundation. The form allows the concrete to be pumped under the eroded footing and displaces the water in the encasement through vents.

23.7.7. Concrete Bridge Seat Repair or Extension

Concrete bridge seats can fail due to deterioration of concrete, corrosion of the reinforcing bars, friction from the beam or bearing devices sliding directly on the seat, and the improper design of the seat which results in shear failure. Anchoring an extension to the existing cap restores adequate bearing for beams that have deteriorated or sheared at the bearing. The extension should not be exposed to any load during curing. Also, repair any damage to the end of the beam itself.

23.7.8. Wingwall Repair

In many old concrete abutments, the wingwalls tend to break-off and to separate from the main body due to earth pressure and differential settlement. If the opening has been stable, the do-nothing option may be acceptable. If not stable, remove the wingwalls and completely rebuild. Footings for the new wingwalls should be at the same level as that of the main body.

23.7.9. Micropile Underpinning

Micropiles, also known as minipiles and pin piles, are small-diameter reinforced piles that are drilled and grouted to support structures. These piles may reach service loads up to 300 tons, can be installed to depths of approximately 200 feet, and usually use some type of steel bar or bars and/or steel casing pipe. The bars are grouted into the ground and/or the casing pipe is filled with grout.

Although a conventional pile is generally quite large and requires heavy equipment and large staging areas for installation, micropiles can be used where conventional piling is not convenient or possible, such as for underpinning or retrofitting existing bridges or structures.

Micropiles have proven effective in many ground improvement applications by increasing the bearing capacity and reducing settlement, especially when strengthening the existing foundations.

23.8. Seismic Retrofit

23.8.1. Seismic Evaluation

The ability to predict the forces developed by an earthquake is limited by the complexity of predicting the ground acceleration and displacement, and the associated response of the structure. The motion can generally be described as independent rotation, in any direction, of each bridge abutment or pier, in or out of phase with each other, combined with sudden vertical displacements. The ground between piers can distort elastically and in some cases rupture or liquefy.

Historically, bridge failures induced by the motions of the abutments and piers stem from two major inadequacies of many existing bridge designs — the lack of adequate connections between segments of a bridge and inadequately reinforced columns. Other deficiencies include inadequately reinforced footing and bent cap concrete and inadequate design force levels considering the likelihood of earthquakes at the location.

Fortunately, tying the segments of an existing bridge together is an effective means of preventing the most prevalent failure mode — spans falling off the supports. It is also the least expensive of the seismic deficiencies to correct. Bridges with single-column piers are particularly vulnerable where segments are not connected.

Column failures have historically been associated with inadequate confinement, shear, and anchorage reinforcement. Confinement and shear deficiencies have occurred where too few and/or improperly detailed ties and spirals are present. Short-lapped splices in longitudinal column reinforcing have resulted in anchorage failures. These failure modes are particularly critical in single-column piers.

Determining the preferred retrofit strategy involves the following considerations:

- anticipated failure mode
- influence on other parts of the bridge under seismic and normal loadings
- interference with traffic
- fabrication and installation cost

Some retrofit strategies are designed to correct bridge inadequacies related to seismic resistance. The procedures may be categorized by the function the retrofit serves, including:

- restraining uplift,
- restraining longitudinal motion,
- restraining hinges,
- widening bearing seats,
- isolating/modifying seismic forces between the superstructures and substructures (seismic isolation bearings),
- strengthening columns and footings, and
- restraining transverse motion.

For seismic retrofits as part of 3R projects, determine the Seismic Performance Zone in accordance with the *LRFD Specifications.* Bridges in Seismic Zone 1 are not required to be retrofitted. Retrofit bridges in Seismic Zones 2, 3, and 4 as needed based on this *Manual* and the FHWA *Seismic Retrofitting Manual for Highway Bridges*, May 1995. Essential bridges will be identified by the Chief Bridge Engineer.

23.8.2. Typical DOT&PF Practices

Highway bridges are vulnerable to partial or total collapse during earthquakes due to three main reasons:

- girders dropping from their supports
- seismic moments and shears exceeding the capacity of the columns
- ground failure associated with liquefaction and lateral spread

Seismic retrofit attempts to improve the performance of existing bridges that are vulnerable to these demands. Retrofits typically fall into one of two categories: Phase I or Phase II.

Phase I Seismic Retrofit

The objective of Phase I retrofit strategies is to prevent girders from falling off their supports (abutment and pier seats) in a relatively cost-effective manner.

Tie the girders to each other and to their supports with restrainer cables to limit their displacement relative to the supports.

Seismic retrofit can also limit girder movement by installing concrete shear keys. On some bridges, installing timber blocking between the ends of the girders and the abutment backwall limits longitudinal girder movement and attracts seismic forces away from the piers to the abutments.

Another strategy involves increasing the support width, thus increasing the displacement capacity of the system.

Phase II Seismic Retrofit

Categorize all work beyond Phase I as Phase II retrofit strategies. These Phase II strategies are generally much more expensive. They typically address column seismic deficiencies through retrofits such as column jacketing or foundation deficiencies through retrofits such as footing modifications.

Seismic isolation is generally categorized as a Phase II retrofit.

23.8.3. Retrofit Techniques

Column Jacketing

Jacketing consists of adding confinement to columns by covering with a grout-filled steel shell, fiberglass wrap, or carbon fiber wrap.

The steel jacket consists of structural steel welded over the column and grouted. The fiberglass and carbon fiber wraps are glued to the column in multiple layers. These wraps are proprietary products.

Non-circular columns can be retrofitted by jacketing, but the increased rigidity must be evaluated. A circular steel casing may be placed around the non-circular column and grouted.

Locate jacketing only at the points of potential column hinge formations. However, if more than half the total height of the column requires a jacket, typically extend the jacket to full height for improved aesthetics.

Jacketing increases column rigidity, amplifying global seismic forces and attracting more load to the column. The bridge engineer typically needs to evaluate this increased rigidity.

Seat Width Enlargement

Seat width extensions allow larger relative displacements to occur between the superstructure and substructure before support is lost, and the span collapses.

The seat width extension strategy at piers is typically selected when the pier has inadequate strength to resist the forces from restrainer cables connecting the superstructure to the pier. The extensions are likely to be exposed to large impact forces due to the dropping span; therefore, they should either be directly supported by a footing or be adequately anchored to the abutment or pier cap.

Follow the provisions in the *LRFD Specifications* relative to the design of seat widths, as practical.

DOT&PF typically uses a combination of post tensioning rods and epoxy-bonded dowels for pier cap widenings.

Post tensioning rods are generally impractical at abutments due to the need to excavate behind the backwall and the resulting disruption of traffic. Seat width dimensions are frequently controlled by the development length of the epoxy-bonded dowels. Increase the vertical load by 100 percent (200 percent of dead load) to account for impact effects.

Structural Continuity

Some older bridges were constructed with multiple simple spans. They lack longitudinal continuity. These older bridges frequently have minimal support lengths under the ends of the girders and limited restraining devices (e.g., anchor bolts).

Historically, limited support lengths, inadequate restraining devices, and lack of superstructure continuity have contributed to spans collapsing during earthquakes.

It is typically not feasible to retrofit a superstructure to make it continuous for full dead and live loads unless the deck is being replaced; however, steel plate girder bridges can sometimes have web-continuity plates added over the piers. These plates and their connections to the existing girder ends should be designed for increased vertical loads (200 percent of dead load reaction) to account for impact effects.

Restrainers and Ties

In general, restrainers are add-on structural devices that do not participate in resisting other than seismic force effects. Typically, these components are made of steel, design them to remain elastic during seismic action, and exercise special care to protect them against corrosion.

There are three types of restrainers — longitudinal, transverse, and vertical. The purpose of the two former types is to prevent unseating the superstructure. The objective of the third type is to preclude secondary dynamic (impact) forces that may result from the vertical separation of the superstructure. The restraint devices should be compatible with the geometry, strength, and detailing of the existing structure.

Ties are restrainers that connect only components of the superstructure together. They are activated only by seismic excitation.

Shear keys or blocks allow in-service movements of the bridge, without applying significant loads to the substructure. During an earthquake, the stoppers transmit the seismic force to the substructure. Transverse stoppers are used frequently in seismically isolated bridges. Longitudinal stoppers are uncommon due to the larger in-service longitudinal movements of bridges.

Steel Rocker Bearings

For bridges within Seismic Zones 2, 3, and 4 (SDC B, C, and D), the retrofitting measures include modification or elimination of existing steel rocker bearings. Major reconstruction projects in Zone 1 (SDC A) may also be good candidates for the elimination of existing steel rocker bearings, which will be decided on a case-by-case basis.

Rocker bearings are replaced with steel reinforced elastomeric bearing pads or seismic isolation bearings. Steel rocker bearings are typically much taller than the replacement devices (e.g., pads, isolators).

This height difference necessitates a reinforced concrete fill be cast between the top of the existing cap beam and the underside of the new bearing devices.

Seismic Isolation Bearings

There is a broad variety of patented seismic isolation bearings that are commercially available. They permit either rotation or translation or both. They have special characteristics by which the dynamic response of the bridge is altered, and some of the seismic energy is dissipated. The primary change in structural response is a substantial increase in the period of the structure's fundamental mode of vibration. The devices are designed to perform elastically in response to normal service conditions and loads.

DOT&PF typically uses only friction pendulum bearings because they are relatively unaffected by temperature changes. Sole source procurement policies need to be followed when specifying these proprietary devices.

See the AASHTO *Guide Specifications for Seismic Isolation Design* and the FHWA *Seismic Retrofitting Manual for Highway Structures*, FHWA, 2006 for more information.

Modifying Seismic Response

Use the following techniques to modify the seismic response of a bridge:

1. **Flexural Reinforcement.** Because of conservative provisions, concrete columns have often been both over-designed and over-reinforced in the past.

   Over-reinforcement means that the flexural steel is not expected to yield during the design event, resulting in both higher compressive and shear forces on the concrete. If other design criteria permit, some of the flexural steel may be cut to induce yield. If circumstances warrant, the flexural reinforcement may be increased.

   Locate the vertical bars in a concrete jacket that is shear connected to the column by means of drilled and grouted dowels. This also increases the rigidity of the column, potentially rendering it counterproductive.

2. **Infill Shear Wall.** A concrete shear wall can be added between the individual columns of pier.

   If the existing footing is not continuous, it should be made so.

   Connect the wall to the columns by means of drilled and bonded dowels. This method substantially changes the seismic-response characteristics of the structure, requiring a complete reanalysis.

   The more rigid infill wall may attract more load, and this increase must be considered in the design.

23.9. Bridge Widening

23.9.1. General Approach

A bridge widening can present a multitude of challenges during the planning and design stages, during construction, and throughout its service life. Pay special attention to both the overall design and detailing of the widening to minimize construction and maintenance problems.

This section presents DOT&PF guidelines for widening existing bridges. The following briefly summarizes the basic objectives in bridge widening:

- Match the structural components of the existing structure, including splice locations.
- Match the existing bearing types in terms of fixity.
- Do not perpetuate fatigue-prone details.
- Evaluate the need to replace the bearings and joints in the existing structure.
- Evaluate the load-carrying capacity of the existing structure.
- Evaluate the seismic resistance of the existing and widened structure. Incorporate retrofit measures if appropriate.
- Use the same structure frame on the widened portion as on the existing bridge.
- Match the flexibility/stiffness of the existing and new superstructures.

23.9.2. AASHTO Standards

It is not normally warranted to modify the existing structure solely because it was designed to AASHTO Specifications prior to the adoption of the *LRFD Bridge Design Specifications* and its latest interim changes.

When preparing plans to modify existing structures, it is often necessary to know the live load and stress criteria used in the original design. With few exceptions, structures on the Alaska highway system have been designed for loads and stresses specified by AASHTO.

The bridge engineer should be aware of the historical perspective of design criteria, such as live loads, allowable stresses, etc., when analyzing a rehabilitated structure. For accurate and complete information on specific structures, see the General Notes on as-built plans, old standard drawings and special provisions, and the appropriate editions of the AASHTO Specifications.

23.9.3. Details of Existing Structures

Load-Carrying Capacity

An existing structure may have been originally designed for either live loads or seismic loads less than those currently used for new bridges.

If such a structure becomes a candidate for widening, consult the data available in the Alaska Bridge Inventory on the condition of the existing structure. Determine a load rating for the existing bridge to quantify the capacity of the existing bridge (see Chapter 27). Based on this information, the bridge engineer will determine whether the existing structure should be strengthened to increase load-carrying capacity. For this evaluation, consider the following:

- cost of strengthening existing structure
- physical condition, operating characteristics, and remaining service life of the structure
- seismic resistance of structure
- other site-specific conditions
- only structure on route that restricts permit loading
- width of widening
- traffic accommodation during construction

Rolled Steel Beams

Throughout the years, modifications to rolled steel beam sections have occurred. Bridge engineers should refer to the construction-year AISC steel tables for rolled beam properties and other data.

Materials

For material properties of older structures, check the General Notes on the existing bridge plans, if they exist, or plans of comparable bridges of the day. Also, reference the MBE for historical properties of materials.

Sometimes, the grade of reinforcing steel is indicated as "intermediate grade"; this terminology means Grade 40.

Until approximately 1960, ASTM A7 was the primary structural steel used in bridge construction. The yield and tensile strengths of A7 may be taken as 33 ksi and 66 ksi, respectively.

23.9.4. Girder Type Selection

In selecting the girder type for a structure widening, the widened portion of the structure should be a construction type and material type consistent with the existing structure. Proportion the widening to ensure that the structural response is similar to the existing bridge.

23.9.5. Deck Closure Pour

Where dead load deflection exceeds ¼ inch, allow the widening to deflect and use a closure pour to complete the attachment to the existing structure.

A closure pour serves two useful purposes: It defers final connection to the existing structure until after the deflection from the deck slab weight has occurred; it provides the width needed to make a smooth transition between differences in final grades that result from design or construction imperfections.

The bridge plans should include a note indicating the required waiting period, if any, between deck concrete and closure concrete placement.

23.9.6. Vehicular Vibration During Construction

Structures deflect when subjected to live loading, and many bridge widenings are constructed with traffic on the existing structure. Fresh concrete in the deck is subjected to deflections and vibrations caused by traffic. Studies such as NCHRP 86 *Effects of Traffic-Induced Vibrations on Bridge-Deck Repairs* have shown that:

- Good-quality reinforced concrete is not adversely affected by jarring and vibrations of low frequency and amplitude during the period of setting and early strength development.
- Traffic-induced vibrations do not cause relative movement between fresh concrete and embedded reinforcement.
- Investigations of the condition of widened bridges have shown satisfactory performance of attached widenings, with and without the use of a closure pour placed under traffic.

23.9.7. Substructures/Foundations

Investigate foundation capacities of existing structures if additional loads will be imposed on them by the widening. Settlement of newly constructed footings under a widened portion of a structure is possible. The new substructure could be tied to the existing substructure to reduce the potential for differential foundation settlements, provided that this does not adversely affect the existing substructure. If the new substructure is not tied to the existing substructure, make suitable provisions to prevent possible damage where such movements are anticipated.

Work with Statewide Materials to assess the compatibility of new and existing foundations and the potential for differential settlement.

23.10. Post-Earthquake Repairs

23.10.1. Post-Earthquake Column Repair

Prepare repairs to columns damaged by earthquakes in accordance with the methodology outlined in the "Repair of Reinforced Concrete Bridge Columns via Plastic Hinge Relocation, Volume 3: Design Guide" by Krish et al (2018). Additional column repair details may be implemented in accordance with "Rapid Seismic Repair of Column to Footing Connections – Phase 2" by Brodbeck et al (2002).

Appendix 23.A

REPAIR OF DAMAGED IN-SERVICE PRESTRESSED CONCRETE BEAMS

23.A.1 Materials[1]

23.A.1.1 Epoxy for Crack Injection

Meet the requirements of ASTM C881 Type IV, Class B or C for epoxy for crack injection.

23.A.1.2 Concrete Bonding Agent

Meet the requirements of ASTM C1059 Type II for bonding agent used for concrete spall repairs.

23.A.1.3 Reinforcing Steel

Conform to *Alaska Standard Specifications*, Section 503 for reinforcing steel. Use ASTM A706 Grade 60 reinforcing steel. Reinforcing steel coating should match that which was originally in the area being repaired. Repair all damaged coatings on salvaged bars.

23.A.1.4 Concrete[2]

For the concrete repair material use a preblended, prepackaged cement-based mortar requiring only the addition of potable water. Do not use material that contains any chlorides or lime other than amounts contained within the hydraulic composition. Use a concrete repair material with sufficient compressive strength as defined by ASTM C109 to reach the original girder's f'_{ci} within the time the preload is applied to the girder. Achieve the bond strength of the concrete repair material when tested according to ASTM C882 that meets the requirements in the table below.

Original Beam 28-day Compressive Strength (f'_c) (psi)	Required Bond Strength (psi)
4000	285
5000	320
6000	350
7000	380
7500	390
8000	405
8500	415

23.A.1.5 Couplers and Tensioning Devices

All couplings must develop at least 95% of the specified minimum ultimate strength of the prestressing steel without permanent deformation. Localized yielding of coupling components is permitted; however, generalized permanent yielding is not permitted. The coupling of tendons must not reduce the elongation at rupture below the requirements of the tendon itself.

23.A.2 Repair Procedures

23.A.2.1 Patching Concrete

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1 Waheed, Kowal, Loo, Page 8

2 Based on currently used patch material as reported by Banse, October 3

Clean all patch joints of surface laitance, curing compound and other foreign materials before fresh concrete is placed against the surface of the joint. Roughen the existing concrete surface to a full amplitude of approximately 1/4 inch by abrasive blasting or mechanical equipment. Do not discharge pulverized concrete and waste sand from abrasive blasting into streams. Remove the pulverized concrete and waste sand from abrasive blasting from work platforms and dispose of properly. Make saw cuts three quarters of an inch deep in adjacent sound concrete to provide a minimum depth of patch.[3] If the concrete is damaged by heat and/or fire, then take cores as directed by the engineer to determine the extent of the concrete damage. Use abrasive blast methods to clean all construction joints to the extent that clean aggregate is exposed. Flush all construction joints with water and allow to dry to a surface dry condition immediately prior to placing concrete. Remove and repair damaged stressing strand. Repair all damaged reinforcing steel by straightening or replacing as needed. Mechanical splice devices may be used as long as concrete cover is not compromised. Welding may not be done unless the original and the new bars are weldable, and welding procedures and certified welders are approved by the engineer. Repair damaged coatings on reinforcing steel according to *Alaska Standard Specifications* Section 503. Apply a concrete bonding agent prior to placing the patch.

Preload the girder before placing concrete patch[4]. Preload should be of a magnitude and location to produce no stress in the bottom face of the repaired girder. Preload should remain on the girder until the patched concrete attains sufficient strength to withstand the stresses of an unloaded girder. This load is generally very close to f_{ci}, the required strength at stress transfer of the original girder. In all cases, use f_{ci}; however, if unloading the girder at a lower strength is desired, calculations can be made to determine the required strength of the patch when unloaded.[5] Test the strength of the patched concrete material in accordance with AASHTO T 22 and T 23. Field cure the test specimens in accordance with AASHTO T 23, Section 10.2.

23.A.2.2 Crack Repair[6]

Cracks less than 0.013 inches in width need not be repaired. Preload should be applied prior to crack injection[7]. Repair shrinkage cracks on the perimeter of the patches in the same manner as other cracks.

Submit a detailed installation plan to the engineer for review and approval. Include at least the following information in the installation plan:

- Manufacturer's written recommendations for product use, surface preparation, temperature requirements, and all other product requirements relevant to use of the system in this particular application
- Schedule
- Temperature control method
- Surface and crack preparation procedure
- Method of applying the seal to concrete surface
- Mixing method
- Injection method
- Curing method and requirements

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[3] Shanafelt, Horn, Page A-53, Waheed, Kowal, Loo, Page 9

[4] Shanafelt, Horn, Page A-24, Waheed, Kowal, Loo, Page 11

[5] The required strength will be less than f_{ci} because the girder will have additional dead load than the weight of the girder used to determine f_{ci}. Additional dead loads include but are not limited to wearing surface, curb, rail, and diaphragms.

[6] *Alaska Standard Specifications for Highway Construction* Section 501-3.17

[7] Shanafelt, Horn, Page A-17

- Quality control program including Quality Control (QC) Inspector's name, qualifications, and manufacturer's written endorsement of the QC Inspector

Provide a manufacturer approved QC Inspector at the site from the time concrete surface preparation begins until the last crack has been injected with epoxy. Provide a written log of the crack repair procedure, prepared by the QC Inspector, to the engineer within 24 hours of completing the concrete crack repairs. All discrepancies between the installation plan and the QC Inspector's report must be corrected within seven days of completing the concrete crack repairs.

23.A.2.2.1 Preparation

Clean all cracks to be filled, so they are free of dust, silt, oil, and all other material that would impair epoxy bonding to the concrete. Use compressed air jets unless the jets will not remove material from within the cracks, in which case, flush the cracks with water under pressure. When flushing water is used, blow all water out of the cracks with oil-free compressed air before injecting epoxy. Do not use metal brushes, acids, or corrosives to clean concrete.

Insert suitable injection ports in the cracks at intervals equal to the thickness (±5%) of the concrete being injected. At the end of a crack, the first port shall be about half this distance from the end. The spacing of the ports shall be adjusted so that the epoxy substantially fills the cracks.

Seal the surface of the crack between ports with tape or other temporary surface sealant which is capable of retaining the epoxy adhesive in the crack during pressure injection and until the epoxy has hardened.

23.A.2.2.2 Injection

Pump the epoxy adhesive into the cracks through the injection ports. The pump, hose, injection gun, and appurtenances shall properly proportion and mix the epoxy and shall be capable of injecting the epoxy at a sufficient rate and pressure to completely fill all designated cracks. Use a suitable gasket on the head of the injection gun to prevent the adhesive from running down the face of the concrete. Keep pumping pressure as low as practicable.

The temperature of the concrete shall be not less than 40°F or greater than 80°F at the time epoxy is injected.

Before starting injection work and at hourly intervals during injection work when requested by the engineer, a three ounce sample of mixed epoxy shall be taken from the injection gun. Should these samples show any evidence of improper proportioning or mixing, injection work shall be suspended until the equipment or procedures are corrected.

The epoxy adhesive shall be forced into the first port at one end of a crack until adhesive runs in substantial quantity from the next adjacent port. The first port shall then be sealed and injection started at the next port. Injection shall then continue from port to port in this manner until the crack is fully injected. For slanting or vertical cracks, pumping shall start at the lower end of the crack. Where approximately vertical and horizontal cracks intersect, the vertical crack below the intersection shall be injected first. Seal the ports by removing the fitting, filling the void with epoxy and covering the area with tape or surface sealant.

Keep the sealing tape and temporary surface sealant in place until the epoxy has hardened.

23.A.2.2.3 Surface Finish

Remove all sealant tape and other temporary surface sealant when no longer required. Remove all spillage of epoxy.

Finish all exposed concrete surfaces to hide all evidence of the crack and epoxy injection system. Match the color, texture and appearance of the undamaged concrete surrounding the crack to the satisfaction of the engineer.

23.A.3 References

1. Waheed, Abdul; Kowal, Ed; Loo, Tom; *Repair Manual for Concrete Bridge Elements, Version 2.0*, Alberta Infrastructure and Transportation, Technical Standards Branch, Bridge Engineering Section, October 11, 2005, Government of the Province of Alberta.
2. Banse, Steve; *Re: Prestressed Girder Repair Procedures*, e-mail, Sent 2006, October 3, 10:08AKDT.
3. Shanafelt, G. O.; Horn, W. B.; *Evaluation of Damage and Methods of Repair for Prestressed Concrete Bridge Members, Preliminary Draft, Final Report* National Cooperative Highway Research Program Transportation Research Board, National Research Council, Project 12-21, December 1984.
4. Harton, Bruce, *Phone Conversation*, Prestressed Supply, Inc., Manufacturer's Representative, October 10, 2006.
5. AASHTO, *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, American Association of State Highway and Transportation Officials.
6. AASHTO LRFR, *AASHTO Manual for Condition Evaluation and Load Resistance Factor Rating (LRFR) of Highway Bridges*, American Association of State Highway and Transportation Officials.
7. AASHTO, *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials.
8. *Alaska Standard Specifications for Highway Construction*, 2020 Edition.
9. Zobel, Robert S.; Jirsa, James O.; *Performance of Strand Splice Repairs in Prestressed Concrete Bridges*, PCI Journal, Volume 43, Issue 6, Nov/Dec 1998.

24. Railroads

24.1. Highway Bridges Over Railroads (Overheads)

24.1. Highway Bridges Over Railroads (Overheads)

24.1.1. Design Policies and Practices

Design highway bridges constructed over railroads to be consistent with the requirements of the following sources:

- FHWA
- AREMA
- Railroad Companies
- AASHTO LRFD Specifications

FHWA

The Code of Federal Regulations (23 CFR 646 Subpart B "Railroad-Highway Projects") prescribes the FHWA policies, procedures, and design criteria for preparing federal-aid projects involving railroad facilities.

AREMA

The American Railway Engineering and Maintenance-of-Way Association (AREMA) provides recommended practices pertaining to the design, construction and maintenance of railway infrastructure. The organization publishes the *AREMA Manual for Railway Engineering*, which has approximately the same status to railroad engineers as the AASHTO *LRFD Specifications* has to highway bridge engineers.

Railroad Companies

The Alaska Railroad Corporation (the Railroad) is the predominant railroad company that operates in Alaska. The Railroad has adopted specific criteria that could impact the structural or geometric design of a highway bridge over a railroad. The most recent version of *Technical Standards for Roadway, Trail, and Utility Facilities in the AARC Right-of-Way* contains Railroad standards for facilities not owned by the Railroad but located within their right-of-way. The White Pass Yukon Railroad, which runs into Skagway, also operates in Alaska.

AASHTO LRFD Specifications

LRFD Article 3.6.5.1 presents criteria for designing bridge abutments and piers over highways or railroads.

For each highway-bridge-over-railroad project, the bridge engineer's responsibility is to evaluate each of the above during project development. Chapter 24 has been organized by project design element and, as applicable, references one or more of the above sources for the information.

24.1.2. Structure Type and Configuration

Chapter 11 of the *Alaska Bridges and Structures Manual* presents DOT&PF criteria on selecting and configuring highway bridge structure types. Specifically for highway bridges over railroads, the following apply:

Span Length/Configuration

The typical span configuration over a railroad is a single-span or three-span bridge. As part of a cost comparison analysis, the bridge engineer should evaluate a short span versus a long span, which eliminates the need for crash walls.

Falsework

The bridge engineer must work with the project manager and the Railroad early in project development to determine minimum clearance requirements.

Skew

Often, highway bridges over railroads must be designed with a skew greater than 30 degrees. These cases require the approval of the Chief Bridge Engineer in accordance with the *Alaska Highway Preconstruction Manual*.

Bridge Alternatives

In lieu of a highway bridge, the bridge engineer should consider a tube to span a railroad.

See Chapter 11 for more information on structure type selection.

24.1.3. Geometrics

Basic Configuration

The following elements determine the basic geometric configuration of the railroad cross section passing beneath a highway bridge:

- number and type of tracks
- potential for future tracks
- drainage treatments
- access/maintenance roadway (if present)

- Railroad right-of-way
- lateral clearances
- vertical clearances

FHWA will not participate in providing additional width for future tracks unless the Railroad has a documented plan for expansion along that rail line.

Review the following articles, the *AREMA Manual* and current Railroad criteria for additional information.

Lateral Clearances

The Appendix to Subpart B of 23 CFR 646 presents FHWA federal-aid participation limits for lateral clearances. These dimensions are not necessarily consistent with AREMA or the Railroad requirements. The following summarizes the FHWA criteria:

1. **Basic Clearance**. FHWA will fully participate in the cost of a 20-foot horizontal distance measured at right angles from the centerline of track at the top of rails to the face of the embankment slope at a height equal to the elevation of the top of the outside rail.
2. **Additional Clearance**. FHWA will participate in lateral clearances greater than 20 feet to:
    a. provide for drainage, if justified by a hydraulic analysis; or
    b. allow adequate room to accommodate special conditions, if the Railroad demonstrates that this is normal practice.
3. **Piers**. Place all piers at least 9.25 feet horizontally from the centerline of the track and, preferably, beyond the drainage ditch.
4. **Multiple Tracks**. For multiple track facilities, all dimensions apply to the centerline of the outside tracks.

Vertical Clearances

See Table 1130-1 in the *Alaska Highway Preconstruction Manual* for DOT&PF vertical clearance criteria for highway bridges over railroads.

The Appendix to Subpart B of 23 CFR 646 presents FHWA federal-aid participation limits for vertical clearances. Realize that these dimensions are not necessarily consistent with AREMA or the Railroad requirements. The following summarizes the FHWA criteria:

1. **Basic Clearance**. FHWA will fully participate in the costs of a vertical clearance of 23.3 feet above the top of rails, which includes an allowance for future ballasting of the railroad tracks.
2. **Additional Clearance**. Vertical clearances greater than 23.33 feet may be approved on a site-by-site basis where justified by the Railroad to the satisfaction of DOT&PF and FHWA. The Railroad's justification for increased vertical clearance should be based on an analysis of engineering, operational, and economic conditions at a specific structure location.
3. **Temporary Vertical Clearance**. For temporary applications, the bridge engineer may reduce the minimum vertical clearance for a highway over railroad to 21 feet upon approval of the Railroad.

Fencing

If the Railroad requires a protective fence across the highway bridge, use on both sides of highway bridges over railroads. Extend the limits of the fence with barrier rail to the limits of the railroad right-of-way or a minimum of 25 feet beyond the centerline of the outermost existing track, whichever is greater.

24.1.4. Control of Drainage from Highway Bridge Deck

Do not allow deck drains to discharge onto railroad right-of-way without the Railroad's permission. Section 16.4 discusses bridge deck drainage.

24.1.5. Construction Requirements

For information on shoring for construction excavations, see the *AREMA Manual for Railway Engineering*.

Use permanent or temporary steel casings in the construction of drilled shafts that are in load influence zones of railroad tracks. Use casings for the entire length of drilled shafts. Determine the required thickness of casings on a case-by-case basis.

24.1.6. DOT&PF Procedures

Right-of-Way

The Right-of-Way Unit within each Region is responsible for coordinating with the Railroad where DOT&PF projects impact railroads. Right-of-Way's responsibilities include obtaining cost estimates for securing agreements with the Railroad for the relocation and adjustment of their facilities, as required for highway construction, and conducting direct negotiations with the Railroad. The Right-of-Way Unit within each Region will develop and process agreements between the Railroad and DOT&PF. The Railroad and DOT&PF have adopted a Master Agreement for any highway work that impacts Railroad right-of-way.

Project Development

Because of the unique nature of highway-railroad grade separations, special coordination is necessary where a railroad alignment and a highway alignment intersect or where these alignments are in close proximity to each other. The bridge engineer must prepare a preliminary design considering the minimum required horizontal and vertical clearances and submit it to the regional project manager. The Region will coordinate with representatives from the Railroad throughout all phases of the project. The regional project manager will provide the Railroad with final bridge plans for final construction approval.

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25. Construction Support

25.1. Responsibilities

The Bridge Section is involved in support activities related to the construction of structural components on highway construction projects. The Engineer-of-Record (bridge designer) is usually responsible for these activities, in coordination with the construction project engineer (the Engineer).

The Engineer has the responsibility and authority for construction contract administration. As single point of contact, the Engineer will forward bridge-related construction questions, Requests for Information (RFIs), submittal reviews, and Design Clarification/Verification Requests (DCVRs) to the bridge designer. Respond to the Engineer as quickly as possible and within the timeframe specified in the contract documents.

All formal correspondence from the Bridge Section is sent to the Engineer and may be forwarded to the Contractor. The Contractor and Engineer must give written permission to the bridge designer to communicate directly with suppliers and fabricators to expedite review and approval of various submittals. Copy the Engineer and Contractor on all correspondence with suppliers and fabricators to keep them informed of the review and approval status.

25.2. Shop and Working Drawings

The Contractor is responsible for submitting working drawings and shop drawings to the Engineer for review. Contractors, suppliers, and fabricators sometimes request modifications to the structural designs to accommodate their methods. Typically, only the Engineer-of-Record is authorized to approve modifications to the structural design (12 AAC 36.195).

25.2.1. Definitions

The following definitions apply to this *Manual:*

Working Drawings: Drawings, diagrams, illustrations, samples, schedules, calculations, and other documents that illustrate the construction of the work, material, equipment, methods, and items necessary to construct the work according to the plans and specifications. The documents are prepared by the Contractor, and once approved, become a part of the contract.

Shop Drawings: Drawings prepared by the Contractor in greater detail than and supplemental to the plans showing dimensions, manufacturing conventions, and special fabrication instructions used to control the fabrication of portions of the final structure. Once approved, these documents become a part of the contract.

25.2.2. Procedures

The following procedures apply to the bridge designer's review of shop and working drawings.

Contractor Submittals

The DOT&PF *Standard Specifications for Highway Construction* provide general requirements for working drawing submittals and the procedures for their submission, review, and approval. The bridge designer will identify additional items of work requiring shop and working drawing submittals and reviews in the Special Provisions.

The *Standard Specifications* specify the number of submittal copies, but this may not be adequate for some projects. The Engineer, bridge designer, and Contractor require copies of the approved shop or working drawings. The supplier/fabricator, design consultant, Statewide Materials Engineer, and other involved entities may also need copies. Specify the number of required submittal copies in the Special Provisions if different from the *Standard Specifications*.

To expedite review, contractors commonly submit shop drawings and other submittals in an electronic format (e.g., PDF files, AutoCAD files). Until a formal Department-wide policy on electronic submittals is issued, the Bridge Section will accept, review and reply to electronic submittals. When direct correspondence with the fabricator or Contractor has been approved, copy the Engineer in all electronic correspondence.

Review

After review of shop drawings or working drawings, the bridge designer will take one of the following actions:

1. **No Corrections**. If everything is correct on the drawings, stamp "APPROVED" on drawings that require the designer's approval such as shop drawings, erection plans, etc. For drawings that do not require the designer's approval, note that the drawings were reviewed for conformance with the contract and recommend approval by the Engineer.

2. **Minor Corrections**. If only minor corrections are required, mark the corrections and stamp "APPROVED AS NOTED" on the drawings that require the designer's approval. For drawings that do not require the designer's approval, note that the drawings were reviewed for conformance with the contract, mark corrections in red, and recommend approval by the Engineer with corrections as noted. The Contractor will not be required to resubmit the plans for further review and approval.

3. **Major Corrections**. If the drawings contain major discrepancies and errors, stamp "RETURNED FOR CORRECTION" on drawings that require the designer's approval. For drawings that do not require the designer's approval, note that the drawings were reviewed for conformance with the contract and they do not meet these requirements, and recommend that the Engineer return them to the Contractor for correction. The Contractor must revise and resubmit the drawings to DOT&PF with the corrections clearly noted.

On consultant-designed projects, the consultant is responsible for reviewing the shop and working drawings, determining their acceptability, and placing the applicable stamp on the drawings as indicated above. If requested by the Engineer, the Bridge Section will independently review the drawings and convey any suggestions to the consultant for inclusion. The Bridge Section will stamp "REVIEWED" on the drawings. The Bridge Section's review is usually concurrent with the consultant's review.

The State Traffic and Safety Engineer and Bridge Section are both responsible for required sign and signal working drawings. Traffic and Safety reviews these submittals for conformance with layout and electrical requirements; the Bridge Section reviews the structural details.

Review Periods

The *Alaska Standard Specifications* require the Contactor to allow time for review and correction of submittals to the Department. The bridge designer typically makes shop and working drawing review a top priority after receiving the documents. Although the contract documents typically allow 30 days for review, most shop drawings should be reviewed and returned within five working days. In some circumstances, the review will need to be further accelerated.

25.2.3. Checklists

Appendix 25A presents standardized shop drawing and working drawing checklists. These checklists are not necessarily consistent with the contract provisions for every project. Complete the checklists applicable to the project.

25.3. Construction Field and Shop Inspections

The Engineer may ask the bridge designer to conduct field and shop inspections during construction. The designer has a unique perspective and knowledge of the structure design, which can help ensure that construction problems are avoided. Therefore, the designer must be responsive when asked to participate in construction inspection related to structural elements.

Responsibilities

The *Alaska Construction Manual* describes the responsibilities of the various DOT&PF units for construction inspections of structural items.

Field Inspections

The Engineer may request that the bridge designer participate in the following field activities:

- attend pre- and post-construction meetings
- attend pre-concreting conferences ahead of planned concrete placement operations
- observe concrete placement
- for cast-in-place structures supported by falsework, observe the construction operation at some point during falsework construction or before the concrete placement is started
- participate in a close-out or "punch list" inspection. The initial inventory inspection required by the National Bridge Inspection Standards (NBIS) can be combined with this close-out inspection.

25.4. Construction Change Orders

The bridge designer should review change orders related to structural items. The objectives of the review are to:

- determine structural adequacy, consistency with design intent, and consistency with other structural details and specifications;
- calculate and verify the quantities and costs; and
- seal the change order according to Section 13.6 of the *Alaska Construction Manual*.

25.5. Value Engineering Proposals

The *Alaska Standard Specifications* allow Contractors to submit Value Engineering Proposals (VEPs) to modify the plans, specifications, or other requirements in the contract to reduce the total cost of construction without reducing design capacity or quality of the finished product. The *Alaska Standard Specifications* present the procedures that a contractor must follow for a VEP, which is processed as a change order.

25.5.1. Bridge Section Review

The Engineer will seek input from the Bridge Section for any VEP related to structural items. In general, the bridge designer who reviews the VEP must recognize that the contract documents represent one solution to accomplishing the project objectives. For a variety of reasons (e.g., equipment, specialized contractor expertise, field conditions), this solution may not be the most economical. In reviewing the proposal, ensure that the proposed structure is at least equal to the safety, functionality, durability, and longevity of the design presented in the contract documents. Comments on deficiencies in the VEP should be specific and as factual as possible.

25.6. Requests for Information

During the advertising period, prospective bidders may submit Requests for Information (RFIs) to DOT&PF. If related to structural items, these RFIs will be forwarded to the Bridge Section for a response.

Transmit the response through the proper authority in the regional office to ensure consistent responses to all potential bidders. Never directly respond or otherwise contact the bidder.

If changes to the contract documents are necessary, coordinate with the Design Project Manager and Regional Contracts Office to initiate an addendum to the contract documents.

During construction, the Engineer may submit Design Clarification/Verification Requests (DCVRs) to the Bridge Section. Respond to these as soon as possible. If the response is verbal, prepare a Record of Conversation and send a copy to the Engineer for confirmation.

25.7. Materials Certification List (MCL)

During the design phase, the Bridge Section provides a list of material items that require review prior to approval and incorporation into the work during construction. These items include concrete mix designs, reinforcing steel mill certifications, reinforcing steel mechanical splice test reports, elastomeric bearing pad test reports, structural steel mill certifications, and similar documents.

When requested, provide a project-specific MCL to the Design Project Manager for incorporation into the contract documents. The Engineer uses the MCL to identify who (e.g., Bridge, Statewide Materials, Regional QA Engineer) will review and approve Contractor submittals.

Appendix 25.A

SHOP DRAWING CHECKLISTS

Appendix 25.A presents the following standardized checklists for each of the following most common shop drawings:

- precast, pre-tensioned concrete girders
- precast concrete members
- structural steel
- reinforcing bar drawings
- bearings
- expansion joints
- bridge railing

The bridge designer must verify compliance with the shop drawing requirements as specified in the *Alaska Standard Specifications* and contract Special Provisions.

Contract No. ______________________________ Date: ____________________________________

Bridge No. ________________________________ Designer(s): ______________________________

PRECAST, PRE-TENSIONED CONCRETE GIRDERS

Are the following items properly included on the shop drawings for precast, pre-tensioned concrete girders?	**Yes**	**No**	**N/A**
1. All dimensions including total length of girder adjusted to accommodate roadway profile grade and elastic shortening.	☐	☐	☐
2. The number and size of all members.	☐	☐	☐
3. The number, size, and type of prestressing strands, their locations, and the forces in these prestressing elements.	☐	☐	☐
4. Girder end details, including size of blockouts, location, and diameter of holes or inserts, and embedded bearing plates.	☐	☐	☐
5. The location and details of lifting devices and support points if the girder will not rest on its bearings while being stored or transported.	☐	☐	☐
6. The location and type of any inserts required for rail posts, utilities, and other attachments. Verify that correct coating is noted.	☐	☐	☐
7. The layout of the casting bed to be used for casting the prestressed girders showing the location of hold-down devices for any harped strands.	☐	☐	☐
8. Methods for providing and controlling required girder camber during casting, transport, and erection.	☐	☐	☐
9. The location and length of any de-bonded prestressing strands.	☐	☐	☐
10. Jacking forces and number of strands.	☐	☐	☐
11. Path of straight and harped strands, including deflecting saddles (details and required number).	☐	☐	☐
12. The details and type of reinforcing steel including bar size, number per girder, total number, length each, total length, total weight, bent bar, bar coating, minimum lap for size of bar used, and grade of bar used.	☐	☐	☐
13. All general notes and construction notes presented in the contract plans reflected in the shop drawings.	☐	☐	☐
14. Concrete compressive strength at release and at 28 days.	☐	☐	☐

Contract No. ______________________________ Date: ____________________________________

Bridge No. ________________________________ Designer(s): ______________________________

PRECAST CONCRETE MEMBERS

Are the following items properly included on the shop drawings for precast concrete members?	**Yes**	**No**	**N/A**
1. All dimensions.	☐	☐	☐
2. The number and size of all members.	☐	☐	☐
3. Reinforcing materials, sizes, dimensions, orientation, and minimum concrete cover.	☐	☐	☐
4. Concrete 28-day compressive strength and mix design identification number.	☐	☐	☐
5. Joint details and materials.	☐	☐	☐
6. All general notes and construction notes presented in the contract plans reflected in the shop drawings.	☐	☐	☐

Contract No. ______________________________ Date: ____________________________________

Bridge No. ________________________________ Designer(s): ______________________________

STRUCTURAL STEEL

Are the following items properly included on the shop drawings for structural steel?	Yes	No	N/A
1. Principal controlling dimensions and materials.			
a. Length of span adjusted to accommodate roadway profile grade.	☐	☐	☐
b. Length, thickness, and width of plates in primary members and splices.	☐	☐	☐
c. Primary dimensions and/or weight per length of rolled shapes.	☐	☐	☐
d. Diameter, specification, and grade of mechanical fasteners and coating on faying surfaces, if required.	☐	☐	☐
e. Specification, grade, and toughness testing requirements for steel components.	☐	☐	☐
f. Size of fillet welds and partial joint penetration welds; appropriate partial and complete joint penetration weld configurations.	☐	☐	☐
2. Web and flange plates of welded plate girders and rolled beams.			
a. Weld designations.	☐	☐	☐
b. Shop groove weld splice locations.	☐	☐	☐
c. Flange and web tapers and haunches (controlling dimensions only).	☐	☐	☐
d. Location of tension zones in welded members.	☐	☐	☐
3. Stiffener and connection plates.			
a. Width, thickness, material grade, and if toughness testing required.	☐	☐	☐

Are the following items properly included on the shop drawings for structural steel?	Yes	No	N/A
b. Weld size and termination details and bolting to web and flange details.	☐	☐	☐
c. Spacing of intermediate stiffeners.	☐	☐	☐
d. Avoiding interference with shop web and flange splice locations.	☐	☐	☐
e. Fit and location of stiffeners.	☐	☐	☐
f. Bolt hole edge distances and compatibility with diaphragm/cross-frame connections.	☐	☐	☐
4. Bolted splices.			
a. Flange and web splice plate thickness and dimensions.	☐	☐	☐
b. Number, size, and spacing of bolts and holes in splice material.	☐	☐	☐
c. Bolt hole edge distances.	☐	☐	☐
d. Fill plates if necessary.	☐	☐	☐
5. Cross-frames and diaphragms.			
a. Member dimensions and orientation.	☐	☐	☐
b. Number and spacing of connection plate bolts and types of holes, especially for slip-critical connections or details required for differential deflections, especially for horizontally curved members.	☐	☐	☐
c. Size, designation and length of welded connections. Weld termination details.	☐	☐	☐
6. Camber and/or mid-ordinate for cambered rolled beams or welded plate girder sections.	☐	☐	☐
7. Procedures and sequence for shop assembly including handling methods.	☐	☐	☐

Are the following items properly included on the shop drawings for structural steel?	Yes	No	N/A
8. Number and spacing of bolts in floor girder and cross girder connections and special attachments (e.g., brackets).	☐	☐	☐
9. Notes and details relative to cleaning and coating.			
a. Corner preparation (if required for cut edges).	☐	☐	☐
b. Cleaning, required surface preparation, and profile depth (if specified).	☐	☐	☐
c. Shop primer: type; manufacturer; wet or dry film thickness; verification of cure before shop application of subsequent coatings; applicable restrictions on field contact (faying) surfaces; any requirements for pre-priming shop contact surfaces before assembly; and designation of any field weld areas to be left unprimed.	☐	☐	☐
d. Field and top coat(s); shop or field; type; manufacturer; wet or dry film thickness; intermediate coat cure times and/or recoating “window” (time) specified by the contract documents or paint manufacturer’s data sheet; any blockout areas where shop top coats are not permitted (e.g., field splices, diaphragm/cross-frame connections, bearings).	☐	☐	☐
10. Designation of material, tension zones, and welds for fracture-critical members (FCMs).	☐	☐	☐
11. Material designation.	☐	☐	☐
12. Incorporation of all necessary revisions into the shop drawings. (*Significant change may require a change order.*)			
a. Errors or discrepancies in the contract plans discovered during shop drawing preparation or review.	☐	☐	☐
b. All construction changes that affect the shop drawings.	☐	☐	☐
c. Fabricator-proposed modifications approved by DOT&PF and contractor.	☐	☐	☐
d. Proposed material substitutions.	☐	☐	☐
13. Framing plan details.			

Are the following items properly included on the shop drawings for structural steel?	**Yes**	**No**	**N/A**
a. Basic span lengths and, where appropriate, transverse girder spacing.	☐	☐	☐
b. Pier and abutment identifications.	☐	☐	☐
c. Orientation of structure (north arrow), skew(s), spot checks of curve or flare geometry, if applicable.	☐	☐	☐
d. Piecemarks indicated for every element, and their relative location is shown to clarify member orientation.	☐	☐	☐
14. Verification of fabricator certification.	☐	☐	☐
15. Compliance with project-specific requirements that may supersede the requirements of this checklist (such as utility attachments, special connections or connection materials, and stage removal and construction).	☐	☐	☐

Contract No. ______________________________ Date: ____________________________________

Bridge No. _______________________________ Designer(s): ______________________________

REINFORCING BAR DRAWINGS

Are the following items properly included on the shop drawings for reinforcing bar drawings?	**Yes**	**No**	**N/A**
1. For each reinforced concrete component, a bar schedule including bar mark, quantity, total length, bar type, and bend details for each representative bar.	☐	☐	☐
2. Representative drawing of each bar type showing variable bend detail dimensions.	☐	☐	☐
3. Drawings of the component detailing the location and orientation of each bar.	☐	☐	☐
4. Material designation for all bars and coatings.	☐	☐	☐

Contract No. ______________________________ Date: ____________________________________

Bridge No. ________________________________ Designer(s): ______________________________

BEARINGS

Are the following items properly included on the shop drawings for bearings?	**Yes**	**No**	**N/A**
1. Location diagram showing the general layout of the structure with the locations and orientation of the bearings.	☐	☐	☐
2. The number, size, and types of all bearings.	☐	☐	☐
3. Plan and elevation views of bearings showing dimensions, tolerances, and fabrication details; details of all components.	☐	☐	☐
4. Bearing fabrication and assembly details; welding details.	☐	☐	☐
5. Material designations and testing requirements are noted.	☐	☐	☐
6. Steel surface preparation and shop coating details.	☐	☐	☐
7. Design calculations conforming to contract documents, if necessary.	☐	☐	☐

Contract No. ______________________________ Date: ____________________________________

Bridge No. ________________________________ Designer (s): ______________________________

EXPANSION JOINTS

Are the following items properly included on the shop drawings for expansion joints?	**Yes**	**No**	**N/A**
1. General layout and dimensions (overall length, skew angle). Orientation of expansion joint components within the joint blockout.	☐	☐	☐
2. The number, size (movement rating), and types of all expansion joints.	☐	☐	☐
3. Plan and elevation views and sections for all components.	☐	☐	☐
4. Material designations for all components; coatings.	☐	☐	☐
5. Installation widths (minimum and maximum) noted including provisions for temperature variations.	☐	☐	☐
6. Manufacturer recommendations for installation methods and procedures. Required attendance of manufacturer's technical representative noted, if applicable.	☐	☐	☐
7. Where applicable, strip seal glands or joint fillers are provided as one continuous piece. Details for method of splicing glands or joint fillers for non-continuous installations.	☐	☐	☐
8. Details for shop and field welding of steel joint components.	☐	☐	☐
9. Supplementary items for modular expansion joints.			
a. Installation sequence and procedures, lifting locations and mechanisms, leveling assemblies details, adjustments for temperature changes, temporary and permanent anchorage to bridges, and shipping and storage requirements.	☐	☐	☐
b. Maintenance plan, parts list, parts replacement schedule, and inspection instructions.	☐	☐	☐
c. Design calculations and fatigue testing conforming to contract documents.	☐	☐	☐

Contract No. ______________________________ Date: ____________________________________

Bridge No. ________________________________ Designer(s): ______________________________

BRIDGE RAILING

Are the following items properly included on the shop drawings for bridge railing?	**Yes**	**No**	**N/A**
1. General layout and dimensions.	☐	☐	☐
2. The number, size, and type for all bridge railings.	☐	☐	☐
3. Material designations for all components; coatings.	☐	☐	☐
4. All applicable dimensioned details with number required			
a. Rail splice details.	☐	☐	☐
b. Post details.	☐	☐	☐
c. Rail cap details.	☐	☐	☐
d. Anchor plate details.	☐	☐	☐
e. Base plate details.	☐	☐	☐
f. Transition connection plate detail.	☐	☐	☐

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26. Bridge Inspection Program

26.1. Federal Bridge Inspection Program

Alaska has approximately 1,000 bridges on public roads and streets excluding federally-owned bridges.

In general, bridges are designed and constructed with the intent of providing a safe structure and a long service life (50 to 75 years). DOT&PF uses stringent design criteria and construction specifications to accomplish this objective. Nevertheless, all structural elements deteriorate over time, sometimes prematurely and if left unchecked, will become deficient.

In addition, increasing traffic volumes and truck weights subject bridges to stresses for which they were not designed. Therefore, a systematic program of periodic bridge inspections is necessary to evaluate condition and functionality, to detect structural problems, and to extend the useful life of bridges.

26.1.1. National Bridge Inspection Standards

The collapse of the Silver Bridge over the Ohio River in 1967 prompted the United States Congress to enact legislation in Title 23 United States Code 151 “National Bridge Inspection Program.”

This legislation established the National Bridge Inspection Standards (NBIS) (23 CFR Part 650, Subpart C), creating a nationwide bridge inspection, load rating, and inventory program. The Federal Highway Administration has promulgated regulations to establish the specific criteria that each state transportation department must meet; i.e., state DOTs are responsible for proper NBIS safety inspection and evaluation for all public bridges located within the geographic boundaries of the state but not within federal lands.

Primary Elements

The following summarizes the primary elements of the National Bridge Inspection Standards:

- NBIS requires the periodic inspection of all highway “bridges” located on public roads and open to public travel. The NBIS (§650.305) and FHWA *Recording and Coding Guide* defines a bridge as:

  “A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.”

  Figure 26-1 provides examples illustrating this definition.

- Title 23 United States Code 101 defines “public road” as “…any road or street under the jurisdiction of and maintained by a public authority and open to public travel.”

- The state DOTs must load rate all bridges to determine their structural capacity. This includes the calculation of both the operating and inventory ratings. The ratings provide an indication of the bridge’s safe load-carrying capacity. This information also assists in the determination of necessary load restriction posting, the issuance of special overload permits, and the scheduling for rehabilitation or replacement. See Chapter 27 for DOT&PF policies, procedures, and practices related to load rating.

- NBIS establishes the basic requirements for each component of a state DOT bridge inspection program.

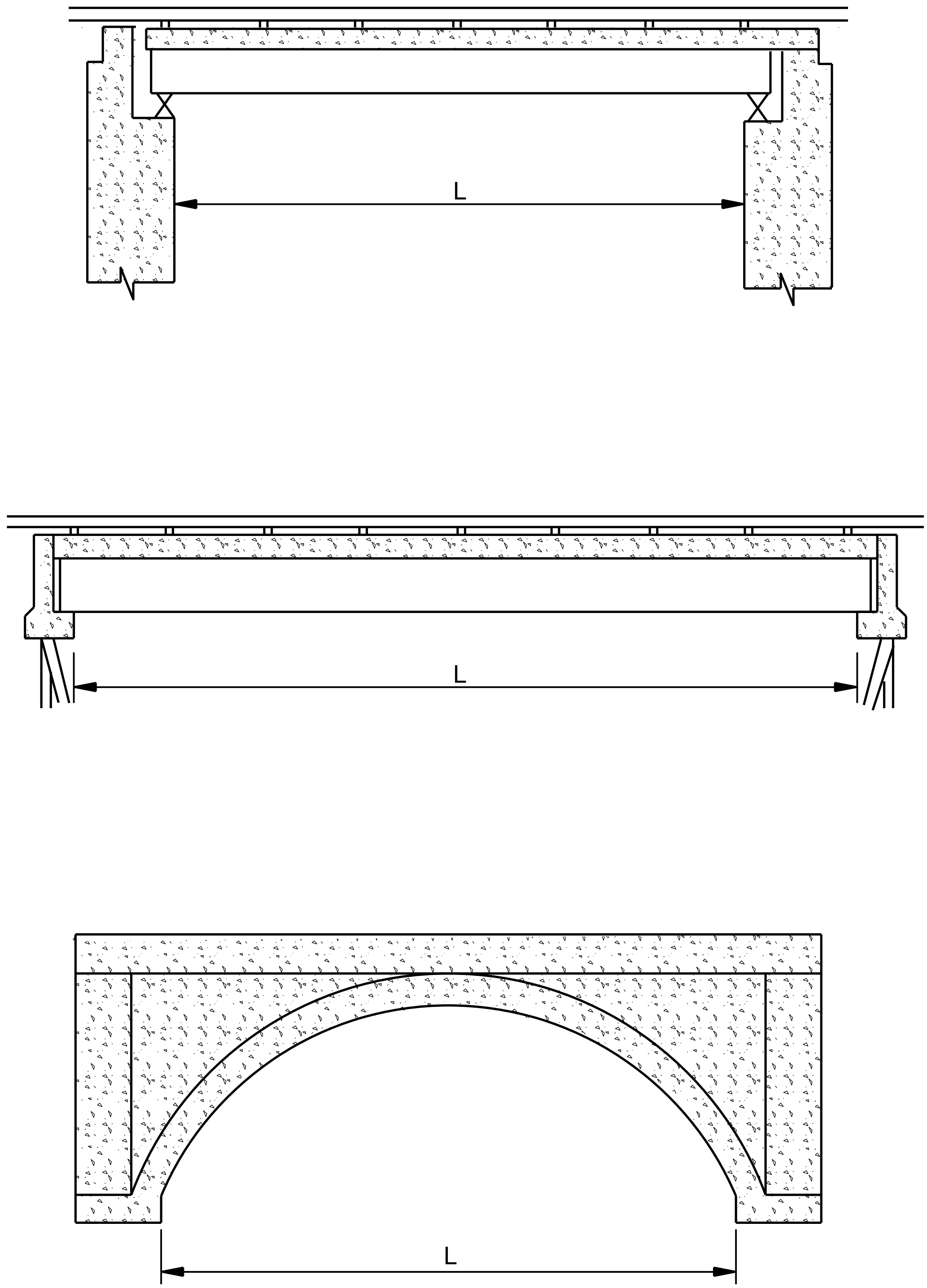
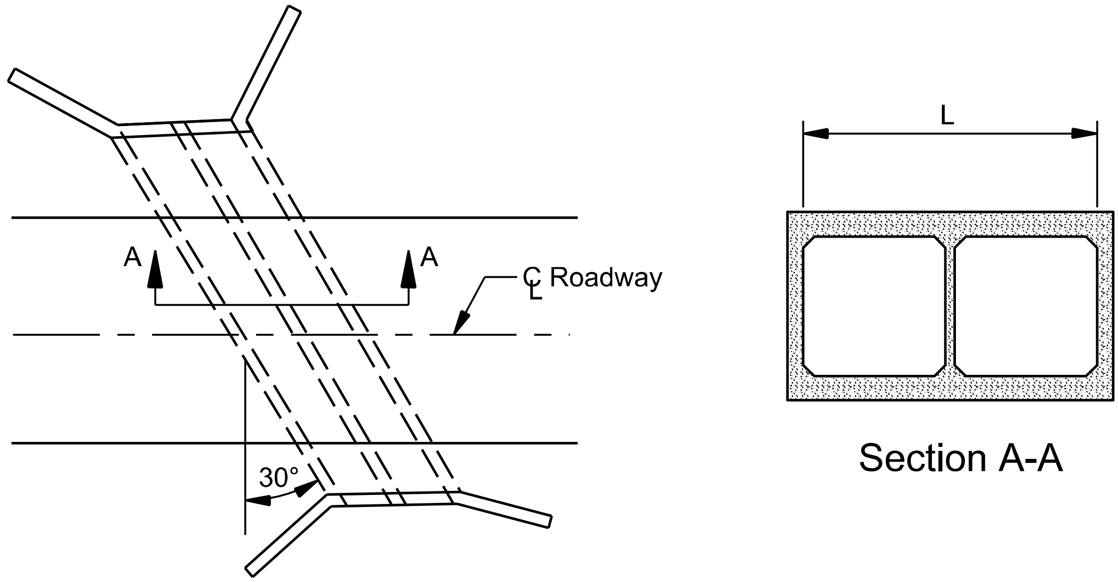


Figure 26-1
NBIS Bridge Length
(Page 1 of 2)



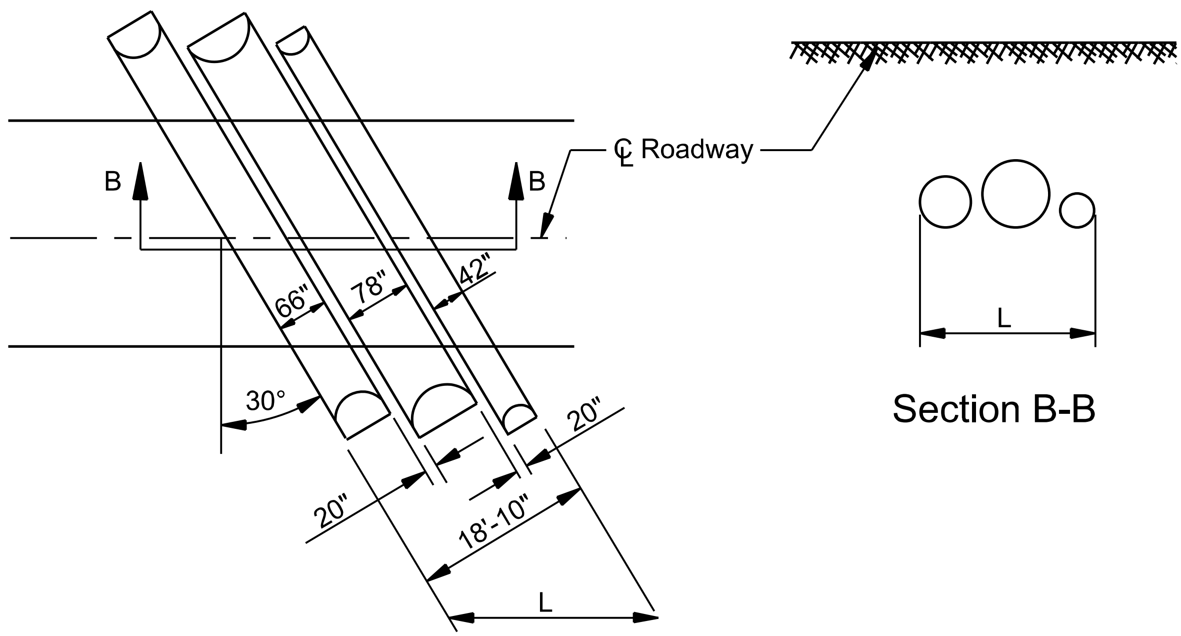


Figure 26-1
NBIS Bridge Length
(Page 2 of 2)

26.1.2. National References

AASHTO and FHWA have developed several references at the national level for the implementation of the NBIS.

FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*

State DOTs use the FHWA *Recording and Coding Guide* when entering specific data items in the National Bridge Inventory (NBI) database. The NBI data is used to prepare legislatively required reports to Congress. The *Recording and Coding Guide* also describes the data necessary for FHWA and the Military Traffic Management Command to identify and classify the Strategic Highway Corridor Network and its connectors for national defense.

FHWA mandates the submission of bridge inventory data to FHWA in a standardized format as required by NBIS. Therefore, DOT&PF has adopted the conventions, terminology, etc., within the FHWA *Recording and Coding Guide* for the collection, recording, and reporting of bridge inspection data.

AASHTO *Guide for Commonly Recognized Structural Elements*

The AASHTO *Guide* provides descriptions of structural elements that are commonly used in highway bridge construction and evaluated during bridge inspections. These elements are termed "Commonly Recognized" (CoRe) structural elements because of their nationwide recognition and use.

Although the element descriptions were originally developed for Pontis (AASHTOWare Bridge Management software), these descriptions were not unique to Pontis; rather, they provided a uniform basis for data collection for any bridge management system and enabled the sharing of data among states.

DOT&PF used the CoRe element descriptions to set up data collection procedures for the DOT&PF bridge management system until October 2014 when the newly adopted AASHTO National Bridge Elements were implemented.

AASHTO *Manual for Bridge Element Inspection*

The AASHTO *Manual for Bridge Element Inspection* supersedes the AASHTO *Guide*, and the CoRe elements were replaced with National Bridge Elements (NBE), Bridge Management Elements (BME), and Agency Developed Elements (ADE). AASHTOWare Bridge Management software (BrM) continues to be used by DOT&PF.

AASHTO *Manual for Bridge Evaluation*

The AASHTO *Manual for Bridge Evaluation* (MBE) serves as a standard and provides uniformity in the procedures and policies for determining the physical condition, maintenance needs, and load capacity of highway bridges in the United States. This publication establishes inspection procedures and load rating practices that meet the NBIS.

The MBE superseded the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*.

The MBE incorporates the load and resistance factor rating (LRFR) methodology plus the traditional allowable stress rating (ASR) and load factor rating (LFR) methodologies.

FHWA *Bridge Inspector's Reference Manual*

The FHWA *Bridge Inspector's Reference Manual* (BIRM) provides guidelines for training bridge inspectors. The BIRM presents a fundamental discussion on the inspection and evaluation of specific bridge components, and it discusses field inspection procedures and reporting requirements. In addition, the BIRM discusses the basic qualifications of bridge inspectors and field safety procedures.

The BIRM is used as a primary reference in the comprehensive training program on bridge inspection presented by the National Highway Institute (NHI).

AASHTO Culvert and Storm Drain System Inspection Guide

AASHTO has accepted but not yet published this *Guide*. The *Guide* provides inspection and asset management information for culverts that updates the 1986 FHWA *Culvert Inspection Manual*. The *Guide* is not intended to be used as a standard or policy statement, but is a resource that provides supplemental information for culvert assessment.

26.1.3. Federal-Aid Program

See Section 28.3.2.

26.2. Alaska Bridge Inspection Program

The Alaska Bridge Inspection Program represents DOT&PF's implementation of the National Bridge Inspection Standards for all public bridges in the state of Alaska not owned by federal agencies, and incorporates data collection to support the state's bridge management system.

Sections 26.3 through 26.7 discuss specific procedures and criteria adopted by DOT&PF for its implementation of the Alaska Bridge Inspection Program. It is the policy of DOT&PF that the provisions for bridge safety inspection and evaluation in this *Manual* meet or exceed the minimum NBIS standards and related FHWA policy.

26.2.1. Coding Bridge Inspection Data

FHWA and AASHTO have developed rating systems to aid bridge inspections. The two primary rating systems currently in use are the National Bridge Inventory (NBI) rating system and the element level rating system. Both rating systems promote uniformity for rating the structural condition of a bridge. The bridge inspector must complete both an NBI and an element level inspection of each bridge inspected.

National Bridge Inventory (NBI) Inspection

The bridge inspector collects NBI data in accordance with the FHWA *Recording and Coding Guide*. The FHWA Supplemental Rating Guidelines for rating decks, superstructures, and substructures may be used. See Appendix 26.A.

An NBI inspection evaluates the deck, superstructure, substructure, channel protection, and culvert adequacy for each bridge, and assigns a single "Condition Rating" representing the condition of the component type as a whole, regardless of quantity. Condition rating codes range from 9 to 0, where 9 is the best rating possible ("excellent condition") and 0 is the worst rating possible ("failed condition").

The Bridge Management Unit incorporates these specific condition ratings into the Structure Inventory and Appraisal (SI&A) sheet, which is a comprehensive listing of all NBI data for any given bridge. FHWA uses the condition ratings, in part, to determine the Sufficiency Rating (SR) for the bridge, to determine eligibility for Federal Highway Bridge Funds, and to allocate these funds to the states.

Element Level Inspection

An element level inspection identifies each bridge component as a separate element, based not only upon function but also material type, and it evaluates each element by sub-dividing its total quantity into different "condition states," or states of physical deterioration or damage. The AASHTO *Manual for Bridge Element Inspection* describes the bridge element-based rating system.

In an element level inspection, each bridge element has an element number and a standard description. The total quantity for each element is then sub-divided among the available condition states, where condition state 1 indicates the best possible condition. Bridge owners use additional agency-developed elements called "smart flags" to track event-driven damage, such as traffic impacts or fatigue cracks that are unique to a specific bridge. The bridge inspector supplements the quantitative condition states with narrative descriptions of their observations. Repair recommendations are presented as "work candidates."

The element level rating system incorporates data that, over time, can be used to estimate deterioration rates based on the structural material and the bridge environment. This allows bridge owners to schedule preventive and corrective actions more uniformly, predict future bridge conditions, and estimate necessary funding to maintain a desired bridge condition. In this way, DOT&PF can make informed decisions to optimize the expenditure of funds and to determine when to take action and what type of action to take. See Chapter 28 DOT&PF policies, procedures, and practices related to bridge management.

26.3. Responsibilities/Qualifications

26.3.1. DOT&PF Bridge Management Unit (§650.307)

The Bridge Management Unit is responsible for:

- ensuring that all bridges subject to the NBIS (excluding federally owned bridges) are inspected and load rated
- developing statewide inspection and load rating policies
- supporting bridge asset management.

26.3.2. Qualifications of Personnel (§650.309)

Bridge Management Engineer

The Bridge Management Engineer serves as the Program Manager (PM) for implementing National Bridge Inspection Standards. The PM meets NBIS qualification requirements. The PM is defined under §650.305 as:

"The individual in charge of the program that has been assigned or delegated the duties and responsibilities for bridge inspection, reporting and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance."

The PM ensures that DOT&PF complies with federal regulations and directives for structure inspection, load rating, and inventory maintenance.

The responsibilities of the PM include:

- Oversee Alaska Bridge Inspection Program including quality assurance and quality control reviews.
- Monitor the inspection program for structures with fracture critical members, underwater members, or unique or complex features requiring additional attention during inspection to assure the safety of such structures; confirm that load-posted structures receive interim inspections as required.
- Review proposals from consultants or contractors to supplement DOT&PF staff, as needed, to perform specialized inspection, testing, or repair. Recommend selected firm and monitors contract performance.
- Recommend coordination actions with federal, state, and local governmental agencies.
- Recommend load posting and bridge closures.
- Develop, monitor, and update training for state and private consultant inspectors.
- Analyze federal and state legislation, administrative rules, and national and industry standards, and recommends implementation into DOT&PF programs and policies.
- Assist in emergency response (e.g., earthquakes, major bridge damage, bridge failures).
- Assist applicable DOT&PF staff in determining appropriate maintenance or repair actions.

Bridge Inspection Manager

The Bridge Inspection Manager assists the Bridge Management Engineer in fulfilling the responsibilities of the Program Manager including the day-to-day management for the Alaska Bridge Inspection Program.

Load Rating Manager

The PM serves as the Load Rating Manager through direct supervision of the individuals who are responsible for calculating bridge inventory and operating ratings, recommending the load posting for existing bridges, and analyzing overweight vehicles for operating permit purposes. §650.309(c) of the NBIS states that:

"The individual charged with the overall responsibility for load rating bridges must be a registered professional engineer."

See Chapter 27 for DOT&PF policies, procedures, and practices related to the responsibilities of the Load Rating Manager.

Team Leader

All inspection team leaders (TL) meet the NBIS qualification requirements and are on-site during field inspections. Team leaders are responsible for reports completed under their direction. §650.305 of the NBIS defines the TL as the:

"Individual in charge of an inspection team responsible for planning, preparing, and performing field inspection of the bridge."

§650.309(b) of the NBIS identifies the qualifications of the TL.

Refresher Training
The PM and all TLs must attend bridge inspection refresher training on a ten-year maximum frequency, with six years being the recommended frequency.

26.3.3. Region Offices

Each regional office has a Maintenance and Operational Director who oversees the maintenance operations in that region. This Director is the primary contact for coordination between the regional office and the Bridge Management Unit for the Alaska Bridge Inspection Program. Regional office involvement in the Program includes the following:

- The region may provide assistance during the bridge inspection as requested by the Bridge Inspection Manager.
- Copies of all bridge inspection reports for state-owned bridges are sent to the appropriate Regional Director.
- The region performs bridge maintenance activities identified in the bridge inspection reports.
- When necessary, the region provides an equipment operator (e.g., boom truck) and traffic control personnel.
- The region reviews and approves the Traffic Control Plan prepared by the Bridge Management Unit.
- Regional personnel may perform initial damage inspections following vehicle impacts, earthquakes, and floods.

26.3.4. Consultant Program

When the state chooses to retain consultants, they are an extension of DOT&PF staff for the implementation of the Alaska Bridge Inspection Program.

During a field inspection, consultant employees must represent DOT&PF professionally in their interface with the public. Typical examples where consultants are retained are:

- fracture critical member (FCM) inspections,
- underwater (UW) inspections,
- NDE inspections, and
- specialized load ratings.

For consultant FCM and UW inspections, the DOT&PF assembles a term agreement or RFP, which presents the specifics of the consultant scope of work.

Operational Issues
In general, consultants must comply with all DOT&PF requirements in implementing the Alaska Bridge Inspection Program. The following discusses a few specific issues:

1. **Engineer in Charge (EIC)**. The EIC is a professional civil engineer registered in the state of Alaska and meets the NBIS team leader requirements and additional time requirements in overall management of the consultant's project. This includes UW and FCM inspections.
2. **Underwater Divers**. Consultants perform all dive inspections. Dive inspectors must meet NBIS qualification requirements for a bridge inspection diver and have additional work experience. The consultant must provide only certified commercial divers.
3. **Submission of Reports**. A professional engineer registered in the state of Alaska must sign and seal final consultant bridge inspection reports prior to submittal to DOT&PF.
4. **DOT&PF QA Review**. The PM or his designated representative will review all bridge inspection reports submitted by consultants for quality, spelling, photograph labeling, completeness, and accuracy. The nature of the DOT&PF review is a quality assurance review, not an "approval."
5. **Load Ratings**. A professional engineer registered in the state of Alaska must sign and seal final consultant prepared load ratings.

26.4. Types of Inspections and Frequencies

26.4.1. General

The following identifies two general parameters for bridge inspections:

1. **Inspection References.** §650.313(a) of the NBIS requires that each state DOT:

   Inspect each bridge in accordance with the inspection procedures in the AASHTO Manual (incorporated by reference, see §650.317).

2. **Inspection Team Composition.** Initial (inventory), routine, in-depth, fracture critical, damage, special, and underwater inspections must have a qualified inspection team leader (TL). The minimum crew size is typically two, including the TL.

26.4.2. Safety

During inspections, evaluate traffic and pedestrian safety features in addition to structural items. Provide special attention to the condition of railings, pedestrian fencing, guardrail, sidewalks, etc. The following are some examples of conditions that may warrant documentation in the bridge inspection report:

- tripping hazards, severe approach roadway settlements, or large spalls on sidewalks
- rebar protruding from decks, walks, or parapets
- loose, missing or damaged railings, or parapets
- missing or damaged guardrail
- loose concrete that could fall onto the traveled way, sidewalk, waterway, or railroad
- any other condition that the inspector perceives as a threat to public safety.

If these conditions are observed during any bridge inspection, they should be documented. See DOT&PF Policy and Procedure 07.05.060 for further information.

26.4.3. Inventory Inspections

§650.305 of the NBIS defines the inventory (or initial) inspection as:

"The first inspection of a bridge as it becomes a part of the bridge file to provide all Structure Inventory and Appraisal (SI&A) data and other relevant data and to determine baseline structural conditions."

An inventory inspection is the baseline inspection that must be completed for every new structure before it can be entered into the Alaska Bridge Inventory. An inventory inspection is a fully documented inspection, using the bridge plans, to determine basic data for a specific structure for entry into the file.

The team leader conducting the inspection must verify quantities and dimensions and must collect the data for the Structure Inventory and Appraisal (SI&A) as required by FHWA regulations.

Inventory inspections are also used when a structure is discovered that has never been inventoried. These inspections are also performed when the configuration or geometry of a structure changes (e.g., widening) or when structural improvements are made that alter previously recorded data (e.g., rehabilitation).

In addition to verifying dimensions and quantities, collect all data required for a routine inspection.

26.4.4. Routine Inspections

§650.305 defines a routine inspection as:

Regularly scheduled inspection consisting of observations and/or measurements needed to determine the physical and functional condition of the bridge to identify any changes from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.

Routine inspections are generally conducted from the deck, ground, or water level or from permanent work platforms and walkways, if present. Inspect the critical load-carrying members (e.g., steel and concrete girders, decks, slabs, piers, bearings, abutments) and closely examine any element that appears distressed. DOT&PF performs routine inspections on a 24-month frequency during the assigned month. Typical data collected during this inspection includes:

- NBI and element data
- vertical clearance or under clearance
- stream cross-section at upstream bridge edge (soundings); and
- information for the Hydraulic Sheet, including an attempt to wade-inspect all substructure members in the water.

The inspection team should take the necessary photographs to visually document the critical aspects of the inspection. Typical photographs, as applicable, are:

1. Ahead at Bridge
2. Back at Bridge
3. Looking Upstream
4. Looking Downstream
5. Near End Load Posting Signs
6. Far End Load Posting Signs
7. Bridge Mounted Signs
8. Typical Deck Condition
9. Elevation
10. Superstructure Underside
11. Abutments
12. Piers
13. Bridge Rail
14. Approach Rail
15. Pin & Hangers
16. In-Span Hinges
17. Seismic Retrofits
18. Stream Bank Condition
19. Stream Bank Protection
20. Typical Streambed Material
21. Utilities

26.4.5. In-Depth Inspections

§650.305 defines an in-depth inspection as:

"A close-up inspection of one or more members above or below the water level to identify any deficiencies not readily detectible using routine inspection procedures; hands-on inspection may be necessary at some locations."

The Bridge Management Engineer determines the need and frequency for in-depth inspections. If follow-up inspections are needed with a frequency of 24 months or less, then the in-depth inspection becomes a special inspection. A routine inspection often identifies conditions that prompt an in-depth inspection. These include:

- need for specialized access,
- need for special inspection/testing techniques and equipment, and
- need for increased inspection of an element.

NBI and element condition data are collected for the members inspected during in-depth inspections.

Thoroughly document the activities, procedures, and findings of in-depth inspections with the appropriate photographs, a location plan of deficiencies, test results, measurements, and a written report. Enter any changes in the condition of the structure into the bridge inspection report, and document any maintenance recommendations.

If a bridge element condition is sufficiently severe, collect sufficient information to load rate the bridge. This inspection data can also be used to develop repair/rehabilitation plans for the bridge.

Description

An in-depth inspection is a close-up visual inspection that often requires special access equipment. Each element under investigation should be within arm's reach of the inspector. The Bridge Management Engineer determines if non-destructive evaluation (NDE) tests and/or other material tests are required. Non-destructive load tests may be conducted to assist in determining a safe load-carrying capacity.

In-depth inspections may also consist of:

- sounding of concrete elements to determine the limits of delamination/deterioration;
- sounding and probing/drilling of timber elements to determine the limits of internal deterioration, rot, and decay;
- connection inspections (bolts, rivets, welds) to identify failing welds/rivets and loose/failing bolts;
- remaining section measurements (as practical) for steel elements; and
- inspection of bearings, paints, or finishes and other miscellaneous structural elements.

26.4.6. Special Inspections

§650.305 defines a special inspection as:

An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency.

The Bridge Management Engineer determines the need and frequency for special inspections. A special inspection may be scheduled when a bridge requires more frequent inspections than is provided by the routine or fracture critical member inspection cycle. A special inspection is typically used to monitor issues that are sufficiently severe to warrant heightened scrutiny, such as foundation settlement, scour, member conditions, and the public's use of a load-posted bridge.

NBI and element condition data are collected for the members inspected during special inspections.

26.4.7. Underwater Inspections

§650.305 defines an underwater inspection as:

"Inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques."

The Bridge Management Engineer determines the need and frequency for underwater and low water inspections. An underwater inspection is scheduled when a bridge inspector cannot easily observe significant underwater structural conditions from above water or at low water until the defect has progressed to where distress is evident. In general, an underwater inspection is required if, during a routine inspection, water conditions exist at the structure that prohibit access to all portions of an element by visual or tactile means that would ensure a level of certainty.

The Bridge Management Unit maintains an underwater master list identifying structures for low-water inspection during low flows and underwater dive inspections. Underwater inspections are performed on a 60-month frequency.

Structural conditions that cannot easily be observed during a routine inspection, but can be seen at low water will receive a low water inspection. Low water inspections are performed on a 60-month frequency from the last time the substructure was inspected.

Structural conditions that cannot be inspected due to unsafe diving conditions may have other means such as acoustic imaging or similar methods used to assess the underwater condition of substructure units.

NBI and element condition data are collected for the members inspected during low water and underwater inspections.

26.4.8. Fracture Critical Member Inspections

§650.305 defines a fracture critical member (FCM) inspection as:

"A hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation."

Section 650.305 defines a FCM as a *"steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse."*

A FCM inspection is a "hands-on" inspection; i.e., made at a distance no greater than arm's length from the entire member/member component surface. NDE may be used to examine potential deficiencies on fracture critical members.

FCM inspections typically require special access to perform a hands-on inspection, and they are completed separate from routine inspections. The Bridge Management Engineer is responsible for determining if a structure member meets the definition of FCM and requires a FCM inspection. The Bridge Management Unit maintains a FCM master list identifying FCM structures and members. FCM inspections are performed on a 24-month frequency, unless a more frequent inspection is necessary to monitor known issues.

NBI and element condition data are collected for the members inspected during FCM inspections.

26.4.9. Complex Bridge Inspections

§650.305 of the NBIS defines a complex bridge as a *"movable, suspension, cable stayed, and other bridges with unusual characteristics."*

§650.313(f) discusses the NBIS requirements for their inspection. Alaska has a few "complex" bridges, including those that are cable-stayed.

The Bridge Management Engineer determines the need and frequency for complex bridge inspections. Complex bridge members will be assessed visually during routine and fracture critical member inspections; otherwise, a special inspection is scheduled.

NBI and element condition data are collected for the members inspected during complex bridge inspections.

26.4.10. Damage Inspection

§650.305 defines a damage inspection as an *"unscheduled inspection to assess structural damage resulting from environmental factors or human actions."*

Common examples of events that may require a damage inspection include earthquakes, floods, vehicular impacts, fire damage, and marine vessel impacts. The Bridge Management Engineer determines the need and frequency for damage inspections. See Section 26.5.1 for the Bridge Section's emergency response procedure.

The scope of the inspection must be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic and to assess the level of effort necessary to implement a repair. The level of effort for a damage inspection can vary significantly and depends on the severity of the damage.

The damage inspection is often followed by an in-depth inspection to better document the extent of damage and the urgency and scope of repairs. Follow-up activities include proper documentation, verification of field measurements and calculations and, perhaps, a more refined analysis to establish or adjust interim load restrictions.

NBI and element condition data are collected for the members inspected during damage inspections.

26.4.11. Inspections of Miscellaneous Structures

Although NBIS does not require the inspection of the following structures, DOT&PF practices for inspecting these miscellaneous structures is:

Tunnels

Tunnels carrying unrestricted highway traffic receive routine inspections every 24 months. The inspection report includes condition ratings, written documentation of findings, and maintenance and rehabilitation recommendations.

MAP 21 requires states to develop a tunnel inspection program.

Mandatory inspection frequency will be determined by future federal regulations.

High-Mast Lighting

DOT&PF performs periodic inspections on these structures on a four- to five-year cycle.

The Bridge Management Unit maintains a master list identifying high-mast lighting requiring inspection. The Bridge Management Unit has developed a special form for inspecting high-mast lighting and a computer program to record the inspection data.

Pedestrian Structures

DOT&PF performs inspections of pedestrian structures over highways that are owned and maintained by public agencies.

Vertical clearance measurements are verified for privately owned pedestrian structures located over public roadways.

Inspections and vertical clearance measurements are performed as part of routine inspection trips on a 24-month frequency.

Other Structures

DOT&PF-owned structures meeting the following criteria are inspected on a four- to five-year frequency:

- Bridge lengths between 10 feet to 20 feet
- culverts with a diameter between 10 feet and 20 feet
- separated pedestrian structures within the highway right-of-way

26.5. Special Bridge Inspection Activities

26.5.1. Emergency Response

For an emergency such as a flood or earthquake, the Bridge Section will assemble an appropriate response team. The typical disaster response procedure is:

1. A regional office or upper management contacts the Chief Bridge Engineer or, in his/her absence, contacts the Bridge Management Engineer or a Design Squad Leader.
2. If the emergency is after hours, the Chief Bridge Engineer will contact the appropriate staff in the Bridge Section to assemble and prepare a response plan. This will most likely result in the formation of a response team for bridge inspection and possibly repair.
3. The response team leader will contact the Bridge Section staff as necessary to form the team and implement the response plan.

Additional emergency response information may be found in the DOT&PF "Incident Field Operations Guide," 2018. This guide includes information on bridge assessments and Department-wide phone numbers. It is available online at http://web.dot.state.ak.us/stwdmno/safety/.

Contingency plans for select essential bridges and other available emergency resources are contained in the "Final Highway Bridge Incident Management Plan Report," 2003.

26.5.2. Bridges with Critical Deficiencies, Policy and Procedure

DOT&PF policy is to provide immediate corrective or protective action to safeguard the traveling public when a bridge is determined to be critically deficient.

This section provides procedures and guidance for bridge inspectors and maintenance personnel for addressing bridges with critical deficiencies. The Chief Bridge Engineer and regional Maintenance and Operations Directors share the responsibility to implement this policy.

A critical deficiency is defined as the existence of a bridge or bridge-related condition that is hazardous and requires *immediate* corrective or protective action to safeguard the traveling public. This is comparable to the definition of "critical finding" in §650.305 of the NBIS.

A structural or safety related deficiency that requires immediate follow-up inspection or action.

Bridge inspectors and the authority responsible for maintaining the bridge (responsible authority) must follow this procedure for bridges with critical deficiencies.

See DOT&PF Policy and Procedure 07.05.060.

26.5.3. Scour

Scour is the movement of channel bed material by the action of the moving water, and it has been the leading cause of bridge failures. This movement may result in degradation, or erosion of material or aggradation, or accumulation of material. Degradation of the channel bed may lead to structure instability, posing an often unseen threat to safety. Scour is generally most severe during periods of high flow. When flows recede to normal levels, the presence of scour is often hidden by silt or debris, making detection difficult.

All routine and underwater inspections include an evaluation of substructure/foundation exposure including the assessment of any foundation undermining found during the inspection. Often foundation undermining can only be found or assessed using divers. Record this evaluation under Item 60 of the NBI.

All routine and underwater inspections also include an evaluation of the waterway beneath and adjacent to the bridge. This evaluation includes an assessment of channel scour in the vertical orientation and channel embankment erosion/lateral channel migration in the horizontal orientation. The evaluation also assesses vegetation intrusion and channel bottom material aggradation adjacent to the bridge, and the effectiveness of channel embankment protective measures (e.g., riprap, slope pavement).

Non-State-Owned Bridges

The Bridge Section sends correspondence to local agencies and other state agencies identifying scour-critical bridges that they own and for which they have maintenance responsibility. The correspondence includes a brief explanation of scour critical, why the bridge is considered scour critical, and possible consequences of being scour critical. DOT&PF notifies the owners of their responsibility to take appropriate actions to address their scour-critical bridge(s). An offer is made to provide information on bridge scour and a summary of possible scour countermeasure options.

The Bridge Section maintains a suspense file mechanism to track and periodically send follow-up correspondence to non-DOT&PF bridge owners notifying them of their bridge scour vulnerabilities and requesting their scour plans of action. The Regional Director is copied on all correspondence.

26.5.4. Bridge Instrumentation Program

The purpose of the bridge instrumentation program is to better understand the shear and moments in existing bridges through a more accurate characterization of transverse live-load distribution than that reflected in the tabulated distribution factors. Revised load ratings are calculated using these observed distributions.

26.5.5. Special Bridge Inspection Practices

See Appendix 26.A for chloride deck testing procedure, deck delamination mapping procedure, rocker bearing special inspection procedure, and supplemental rating guidelines.

26.6. Bridge Inspection and Inventory Procedures

26.6.1. Bridge Inspection Document Filing

File original bridge inspection reports in the files located in the "Vault" at the front of the office with the following exceptions.

Bridge Inspection Reports

When hard copy reports are large, place a copy of the cover sheet in the bridge file in the vault with instructions indicating where the complete report is stored. Place electronic copies of all bridge inspection reports in the appropriate bridge file in the Bridge Section "e-vault."

Hydraulic Information

File flood inspection reports in the bridge file(s) in the vault. Flood data may be included in flood inspection reports by state or USGS personnel and are stored in the hard copy bridge folder, e-vault, hard copy hydraulic files, or the "hydro" folder within the Statewide Division's server network.

Waterway adequacy is noted in the SI&A sheets, which are included as part of routine bridge inspection reports.

Channel cross sections are included in routine and scour inspection reports.

File underwater inspection reports in the bridge file in the vault, and file an electronic copy in the e-vault.

File scour data, scour assessments, and USGS hydraulic related reports in the hydraulic files.

The DOT&PF scour Plan of Action (POA) is located in the "hydro" folder within the Statewide Division's server network and maintained by the Bridge Section hydraulic staff.

26.6.2. Scheduling

Section 26.4 presents the frequencies for bridge inspections as required by NBIS. Table 26-1 presents the specific months that DOT&PF schedules routine bridge inspections for locations and even and odd years. Inspection year and month for fracture critical member and underwater inspections are in contained master lists maintained by the Bridge Management Unit.

26.6.3. Office Pre-Inspection Procedures

The team leader (TL) should perform the following office procedures to prepare for the bridge inspection:

Preparation

Review previous bridge inspection reports to identify items requiring special equipment or emphasis.

Equipment

Coordinate all equipment checkouts with the Bridge Inspection Manager.

Determine the inspection equipment that will be needed for the group of bridges that will be inspected and where this equipment is located. Refer to the standard equipment checkout list.

Make the necessary arrangements to relocate the equipment to where it is needed.

Make arrangements for NDE examination and traffic control if applicable.

Where needed, secure the hydraulic equipment (e.g., boogie board, truck-mounted sounding equipment).

Verify that the laptop has the appropriate inspection programs and data.

Coordination

Where possible, notify local agency owners of scheduled inspection dates.

Coordinate inspections on the Yukon River with the Bridge Inspection Manager and Alyeska. Contact Alyeska Security (907-451-5827) and Pump Station 6 (907-787-4606) a minimum of six weeks in advance of the inspection.

Visit local M&O facilities to get local observations and insights regarding local bridges.

Table 26-1
Schedule for Routine Inspections

	Even Year Inspection Trips	Month
1	Alaska Highway	July
2	Aleutians/Kodiak	September
3	Cordova	August
4	Dalton Highway	July
5	Elliott/Steese Highway	August
6	Fairbanks	August
7	Haines/Skagway/Yakutat	June
8	Matsu	June
9	Nome	August
10	North Parks Highway	July
11	South Parks Highway	June
12	Western Outlying	July

	Odd Year Inspection Trips	Month
1	Anchorage	August
2	Denali/North Richardson	July
3	Glenn Highway	July
4	Juneau	June
5	Ketchikan/Hyder	June
6	North Seward/Whittier	June
7	Prince of Wales	July
8	Sitka, Gustavus, Petersburg	May
9	South Richardson	August
10	South Seward/Chenega	August
11	Sterling Highway	August
12	Tok Cutoff/Taylor Highway	July

Railroad. DOT&PF must coordinate with the Alaska Railroad Corporation (ARRC) when its inspection team will be working within 20 feet of the centerline of tracks or when an under bridge inspection vehicle is working over tracks. If necessary, contact AARC Chief Dispatcher (907-265-2421) a minimum of two weeks in advance of the inspection.

Railroad Bridges over Public Roads. Take vertical clearance measurements. If damage is observed, contact the AKRR Chief Dispatcher (907-265-2421) or General Supervisor for Bridges and Buildings (907-265-2541).

Access Keys. The bridge inspector will need to acquire the keys to access fenced areas of bridges or inside of box girders. For DOT&PF-owned bridges, this will be the local maintenance station; for locally owned bridges, this will be the local agency.

Where necessary, coordinate with local maintenance stations for transportation.

26.6.4. Field Inspection Procedures

The following are recommended field procedures for inspections with specialized access equipment or traffic control:

1. **Safety Briefing.** When non-DOT&PF personnel are present, brief them on DOT&PF safety requirements before starting the inspection. Do not proceed with any inspection without the proper personnel being present and having received a safety briefing.

2. **Inspection Plan Review.** Examine the detailed inspection plan to determine where to position equipment. Modify the inspection plan by noting the location of piers, abutment slopes, and any other obstructions under the bridge.

3. **Equipment Check.** Verify that the necessary equipment has been assembled and is on site.

4. **Traffic Control.** The team leader is responsible for obtaining an approved Traffic Control Plan.

Bridge-specific inspection procedures, if applicable, are located in the inspection report, bridge file, or both for the subject bridge and inspection type.

Complex bridge inspection procedures for inspection types occurring on a frequency greater than 5 years should be reviewed prior to the inspection to ensure that the most current procedures are used.

Bridge Inspection Safety Task Analysis

See Appendix 26.B.

26.6.5. Office Post-Inspection Procedures

Return all inspection equipment to the Bridge Inspection Manager immediately upon returning to the office. Return the laptop to the database manager. Notify the inspection and/or database manager of any equipment or computer problems. Notify the Bridge Management Engineer or Chief Bridge Engineer of any significant findings or issues encountered during the inspection trip.

26.6.6. Inspection Reporting Procedures

§650.313 presents the NBIS bridge inspection reporting procedures. The following sections present specific DOT&PF reporting procedures for the Alaska Bridge Inspection Program.

Report Preparation

The bridge inspection report incorporates the results of the NBI inspection and element inspection comments and serves as the permanent inspection record. This report portrays the condition of the bridge as it relates to public safety and is used for future rehabilitation and replacement decisions. Therefore, it is imperative that the report presents accurate and thorough information. Reports should include photographs, sketches, addenda, etc., as necessary to adequately and thoroughly document the condition of the structure but also be as concise as possible. Include only information that is necessary to communicate the nature of the structure's condition.

Review and Processing

Use the following for processing bridge inspection reports developed by state personnel:

1. **Field Notes.** Field notes should be reviewed at the inspection site for completeness and accuracy.

2. **Data Entry.** With few exceptions, the TL or Inspection Assistant will enter the bridge inspection data into the appropriate computer program while at the bridge site.

3. **Draft.** The Bridge Management Unit prints draft reports, which are reviewed both by the TL and Inspection Assistant for completeness and accuracy. Rectify all errors and omissions.

4. **QC Review.** The Program Manager or a designee will review all reports generated.

5. **Corrections.** The draft reports are returned to the TL for corrections, which will be revised as necessary.

6. **Final Report.** The Bridge Management Unit prints final corrected reports. The TL and Inspection Assistant initial corrected final reports.

7. **Distribution and Filing.** The Bridge Management Unit copies and distributes final reports. One copy will be filed in the DOT&PF bridge inventory files and e-vault with two copies distributed to the Regional Director or the owner.

Submittal Time Requirements

§650.315(b) of the NBIS stipulates that states have 90 days following the date of inspection for state-owned bridges (180 days for non-state-owned bridges) to update the state bridge inventory. This is required following a field inspection, or at any other time that there is a bridge modification that alters previously recorded data. §650.315(c) of the NBIS stipulates that states have 90 days following the completion of work for new state-owned bridges (180 days for non-state-owned bridges) to update the state bridge inventory. NBIS inventory dates are completed when the TL enters inspection report corrections into BrM.

26.6.7. Bridge Inventory

Definitions

§650.315 (a) of the NBIS requires states to prepare and maintain an inventory of all bridges subject to the NBIS. The following definitions apply:

National Bridge Inventory (NBI): The aggregation of structure inventory and appraisal data collected to fulfill the requirements of the National Bridge Inspection Standards.

National Bridge Inventory Record: Data that has been coded according to the FHWA *Recording and Coding Guide* for each structure carrying a highway.

Structure Inventory and Appraisal (SI&A) Sheet: The report of data recorded and stored for each NBI record in accordance with the FHWA *Recording and Coding Guide*.

NBI Data Reporting

The state annually submits the NBI data to FHWA. The submission to FHWA is typically due by March 31 of each year.

Bridge Inventory Report

The Bridge Management Unit periodically prepares a bridge inventory report for all publically owned bridges in the state of Alaska that are not owned by the federal government. The report summarizes the structural, dimensional, and location data for bridges and culverts that are biennially inspected by the Bridge Section. See the DOT&PF Bridge Section website.

26.7. Quality Control/Quality Assurance

§650.305 of the NBIS defines quality assurance (QA) as:

"The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program."

§650.313(g) requires that a state DOT have QA procedures that:

"Include periodic field review of inspection teams, periodic bridge inspection refresher training for program managers and team leaders, and independent review of inspection reports and computations."

See Appendix 26.C.

Appendix 26.A
Special Bridge Inspection Practices

26.A.1 Chloride Deck Testing Procedure

1. Setup traffic control.
2. Chain drag deck, sketch delaminations and spalls, and select test hole locations with pachometer.
3. Follow procedures per *Federal-Aid Highway Program Manual* and “SHRP.”
    - Collect 1 sample every 500 sq ft (min.) but not less than 10 per structure.
4. Remove surface contaminants: Drill a ¼-inch deep hole with a 1¼-inch diameter drill bit.
5. Clean concrete dust from drill site with compressed air.
6. Place wind shield (cardboard box) around drill site.
7. Drill a 1-inch deep hole with a 1¼-inch diameter drill bit.
8. Collect sample with plastic sundae spoon into a vial and label (5 grams min.).
9. Clean concrete dust from drill site and bit with compressed air and clean spoon.
10. Measure depth of sample hole and record.
11. Drill another ½-inch to 1-inch more with a 1-inch diameter drill bit in the same hole.
12. Collect sample (5 grams min.) with plastic sundae spoon into a vial and label.
13. Clean concrete dust from drill site with compressed air.
14. Measure depth of sample hole and record.
15. Mix cementitious material and fill test holes.
16. Change traffic control in opposite lane.
17. Send sample vials to a testing laboratory, such as:

    CTL Group, Attn: Cyler Hayes
    5400 Old Orchard Road, Dock B
    Skokie, IL 60077-1030

Equipment List (alphabetically):

- Chain drag
- Compressed air
- Ear plugs & eye protection
- Drill bits
- Data collection sheet
- Generator
- Measuring tapes
- Mixing bucket
- Paint sticks & cans
- Pencils & felt-tip markers
- Rebar locator (Pachometer)
- Rite-in-the Rain book
- Rotary hammer
- Set 45
- Spoon(s) (less than 1-inch diameter)
- Traffic control
- Vials, 40 dram size or larger, locking caps
- Wind shield (bottomless packing box)
- Water

26.A.2 Deck Delamination Mapping Procedure

1. Setup traffic control.
2. Chain drag dry deck.
3. Identify delaminated concrete, patch repairs, exposed reinforcing steel, and spalls by using colored spray paint for each area.
4. Measure the length and width of each area.
5. Map out and establish an identification key by lane and span.

Record the following to the nearest 5 feet:

- Total deck surface area to the nearest square foot.
- Total exposed reinforcing steel length.
- Estimated deck surface area of spalls.
- Estimated deck surface are of sound patches.
- Estimated deck surface area of unsound patches.
- Estimated deck surface area of delaminated concrete.
- Estimated defective areas per span using a span-by-span, lane-by-lane table such as below:

Sample Table of a Lane

SPAN	1	2	3	4	5	6	7	Total
Spalls (SF)	150	100	100	150	150	30	20	400
Sound Patch (SF)	50	40	30	60	50	10	5	250
Unsound Patch (SF)	5	0	5	5	5	—	—	20
Delaminations (SF)	100	40	50	100	50	10	20	400
Exposed Rebar (FT)	40	20	60	60	100	—	—	300
Remainder (SF)	2100	2250	2250	2120	2130	2380	2370	15,950

Additional notes (example):

- ***Note:*** *Over one-half of the deck damage is located in the wheel tracks.*

Equipment List (alphabetically):

- Chain drag
- Measuring tape
- Measuring wheel
- Paint roller sticks and spray cans
- Pencils and felt-tip markers
- Rite-in-the-Rain book
- Traffic control devices

26.A.3 Rocker Bearing Special Inspection Procedure

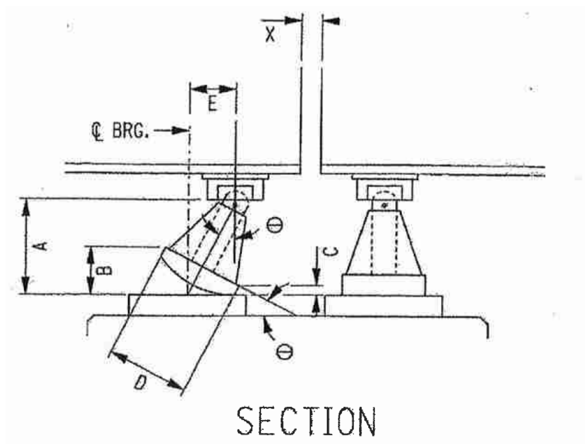
The inspection of rocker bearings is intended to ensure their stability. Compromised (i.e., tipping) bearings could become unstable with relatively small additional deformation or tipping. Use the following procedure to inspect rocker bearings:

1. **Inspection.**

   - Clean dirt and debris off the bearings.
   - Measure tilt of rocker bearings:
     - Hang a plumb bob for reference.
     - Place folding ruler for scale.
     - Mark superstructure for location.
     - Position camera a fixed distance from bearing. Use a 2-foot distance.
     - Take picture with camera the same height as the center of the pin.

2. **Evaluation.** Measure the angle of rotation graphically on the photographs and evaluate. Determine if the vertical line of force is in the middle half of "D" (rocker bearing width). Determine the following:

   - If the vertical line of force is within the middle half of "D," then the bearing is acceptable.
   - If the vertical line of force is outside the middle half of "D," then additional evaluation is necessary.



26.A.4 FHWA Supplemental Rating Guidelines

The following guidelines have been developed as a training guide for the condition rating of a variety of structural elements. They are suggested as a supplement to the FHWA *Recording and Coding Guide* to make it easier to assign the most appropriate condition rating.

26.A.4.1 Timber Deck Condition Rating

Code	Description
9	EXCELLENT CONDITION. No noticeable or noteworthy deficiencies that affect the condition of the deck.
8	VERY GOOD CONDITION. No crushing, rotting, or splitting. Tightly secured to floor system.
7	GOOD CONDITION. Minor checking or splitting with a few loose planks.
6	SATISFACTORY CONDITION. Some of planks checked or split but sound. Some loose planks. Fire damage limited to surface scorching with no measureable section loss. Some wet areas noted.
5	FAIR CONDITION. Numerous (30%-40%) planks checked, split, rotted, or crushed. Many planks are loose. Fire damage limited to surface charring with minor, measurable section loss. Some planks (<10%) are in need of replacement.
4	POOR CONDITION. Majority (over 40%) of the planks are rotted, crushed, or split. Fire damage with significant section loss that may reduce the load carrying capacity of the member. Over 10% of the planks are in need of replacement.
3	SERIOUS CONDITION. Severe signs of structural distress are visible. Extensive plank damage evident with reduced deck load carrying capacity.
2	CRITICAL CONDITION. Advanced deterioration with partial deck failure. May be necessary to close bridge until corrective action is taken.
1	“IMMINENT” FAILURE CONDITION. Bridge is closed. Corrective action may put back in light service.
0	FAILED CONDITION. Bridge closed. Deck replacement necessary.

26.A.4.2 Concrete Deck Condition Rating

	Condition Indicators (% deck area)			
Rating	**Spalls**	**Delaminations**	**Electrical Potential**	**Chloride Content (lbs/cy)**
9	None	None	0	0
8	None	None	None > 0.35	None > 1.0
7	None	< 2%	0-5% > 0.35	None > 2.0
6	< 2% spalls or sum of all deteriorated and/or contaminated deck concrete < 20%			
5	< 5% spalls or sum of all deteriorated and/or contaminated deck concrete 20% to 40%			
4	> 5% spalls or sum of all deteriorated and/or contaminated deck concrete 40% to 60%			
3	> 5% spalls or sum of all deteriorated and/or contaminated deck concrete > 60%			
2	Deck structural capacity grossly inadequate			
1	Deck has failed completely; repairable by replacement only.			
0	Holes in deck; danger of other sections of deck failing.			

26.A.4.3 Steel Grid Deck Condition Rating

Code	Description
9	EXCELLENT CONDITION. No noticeable or noteworthy deficiencies that affect the condition of the deck.
8	VERY GOOD CONDITION. Tightly secured to floor system with no rust.
7	GOOD CONDITION. Loose at some locations with minor rusting. A few cracked welds and/or broken grids.
6	SATISFACTORY CONDITION. Moderate rusting evident with indications of initial section loss. Loose at many locations. Some cracked welds and/or broken grids.
5	FAIR CONDITION. Considerable rusting with some areas of minor section loss. Loose at numerous locations. Numerous cracked welds and/or broken grids.
4	POOR CONDITION. Heavy rusting, resulting in considerable section loss and some holes through deck. Many welds cracked and/or girds broken.
3	SERIOUS CONDITION. Severe signs of structural distress are visible. Repair plates missing with some panel replacement necessary.
2	CRITICAL CONDITION. Many holes through deck.
1	“IMMINENT” FAILURE CONDITION. Bridge is closed. Corrective action may put back in light service.
0	FAILED CONDITION. Bridge closed. Deck replacement necessary.

26.A.4.4 Timber Superstructure Condition Rating

Code	Description
9	EXCELLENT CONDITION. New condition.
8	VERY GOOD CONDITION. No noteworthy deficiencies that affect the condition of the superstructure.
7	GOOD CONDITION. Minor decay, cracking, or splitting of beams or stringers at non-critical locations.
6	SATISFACTORY CONDITION. Some decay, cracking, or splitting of beams or stringers. Fire damage limited to surface charring with minor, measureable section loss.
5	FAIR CONDITION. Moderate decay, cracking, splitting, or minor crushing of beams or stringers. Fire damage limited to surface charring with minor, measurable section loss.
4	POOR CONDITION. Extensive decay, cracking, splitting, or crushing of beams or stringers, or significant fire damage. Diminished load carrying capacity of members is evident.
3	SERIOUS CONDITION. Severe decay, cracking, splitting, or crushing of beams or stringers, or major fire damage. Load carrying capacity is substantially reduced. Local failure may be evident.
2	CRITICAL CONDITION. Beam defects noted in Code 3 have resulted in significant local failures. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	“IMMINENT” FAILURE CONDITION. Bridge is closed. Corrective action may put back in light service.
0	FAILED CONDITION. Bridge closed. Replacement necessary.

26.A.4.5 Reinforced Concrete Superstructure Condition Rating

Code	Description
9	EXCELLENT CONDITION. New condition.
8	VERY GOOD CONDITION. No noteworthy deficiencies that affect the structural capacity of members.
7	GOOD CONDITION. Some minor problems. Non-structural hairline cracks without disintegration may be evident. Load carrying capacity of structural members unaffected.
6	SATISFACTORY CONDITION. Structural members show some minor deterioration or collision damage. Hairline structural cracks or spalls may be present with evidence of efflorescence. Minor water saturation marks. Generally, reinforcing steel unaffected.
5	FAIR CONDITION. Structural members are generally sound (structural capacity unaffected) but may have evidence of deterioration or disintegration. Numerous hairline structural cracks or spalls may be present with minor section loss of reinforcing steel possible.
4	POOR CONDITION. Extensive disintegration. Measurable structural cracks or large spall areas. Corroded reinforcing steel evident with measurable section loss. Structural capacity of some structural members may be diminished.
3	SERIOUS CONDITION. Serious deterioration and/or disintegration of primary concrete members. Large structural cracks may be evident. Reinforcing steel exposed with advanced stages of corrosion. Local failures or loss of bond possible.
2	CRITICAL CONDITION. Advanced deterioration of primary structural elements. Concrete disintegration around reinforcing steel with loss of bond. Some reinforcing steel may be ineffective due to corrosion or loss of bond. Numerous large structural cracks may be present. Localized failures of bearing areas may exist. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	“IMMINENT” FAILURE CONDITION. Bridge is closed to traffic. Major deterioration or section loss present on primary structural elements, obvious vertical or horizontal movement is affecting the structure’s stability. Corrective action may put back in light service.
0	FAILED CONDITION. Bridge is closed: out of service. Beyond corrective action: replacement necessary.

26.A.4.6 Prestressed Concrete Superstructure Condition Rating

Code	Description
9	EXCELLENT CONDITION. New condition.
8	VERY GOOD CONDITION. No problems noted.
7	GOOD CONDITION. Non-structural cracks less than 0.015 inch in width may be evident. No rust stains apparent.
6	SATISFACTORY CONDITION. Minor concrete damage or deterioration. Non-structural cracks over 0.015 inch. Isolated and minor exposure of mild steel reinforcement may be present.
5	FAIR CONDITION. Isolated and minor exposure of prestressing stands may be present. Structural cracks with little or no rust staining. Primary members sound, but may be cracked or spalled.
4	POOR CONDITION. Moderate damage or deterioration to concrete portions of the member exposing reinforcing bars or prestressing strands. Possible bond loss. Structural cracks with medium to heavy rust staining may be present. May be loss of camber.
3	SERIOUS CONDITION. Severe damage to concrete and reinforcing elements of the member. Severed prestressing strand(s) or strand(s) are visibly deformed. Major or total loss of concrete section in bottom flange. Major loss of concrete section in the web, but not occurring at the same location as concrete section loss in the bottom flange. Horizontal misalignment to member or negative camber. Unless closely monitored, it may be necessary to restrict or close the bridge until corrective action is taken.
2	CRITICAL CONDITION. Critical damage to concrete and reinforcing elements of member. This damage may consist of one or more of the following: • Cracks extend across the bottom flange or in the web directly above the bottom flange damage that are not closed below the surface damage. (This indicates that the prestressing strands have exceeded yield strength.) • An abrupt lateral offset as measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength.) • Loss of prestress force to the extent that calculations show that repair cannot be made. • Excessive vertical misalignment. • Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface damage. (This indicates permanent deformation of stirrups.)
1	“IMMINENT” FAILURE CONDITION. Critical damage requiring the replacement of a member. Bridge is closed to traffic and installation of temporary falsework to safeguard the public and the bridge should be taken at the time of the inspection.
0	FAILED CONDITION. Bridge closed and out of service.

26.A.4.7 Steel Superstructure Condition Rating

Code	Description
9	EXCELLENT CONDITION. New condition.
8	VERY GOOD CONDITION. No noticeable or noteworthy deficiencies that affect the condition of the superstructure.
7	GOOD CONDITION. Some rust may be evident without any section loss.
6	SATISFACTORY CONDITION. Rusting evident, but with minor section loss (minor pitting, scaling, or flaking) in critical areas.
5	FAIR CONDITION. Minor section loss in critical areas. Fatigue or out-of-plane distortion cracks may be present in non-critical areas. Hinges may be showing minor corrosion problems.
4	POOR CONDITION. Significant (measurable) section loss in critical areas. Fatigue or out-of-plane distortion cracks may be present in critical areas. Hinges may be frozen from corrosion. Load carrying capacity of structural members affected.
3	SERIOUS CONDITION. Severe section loss or cracking in a critical area. Minor failures may have occurred. Significant weakening of primary members evident.
2	CRITICAL CONDITION. Severe section loss in many areas with holes rusted through at numerous locations in critical areas.
1	"IMMINENT" FAILURE CONDITION. Bridge closed. Corrective action may put back in light service.
0	FAILED CONDITION. Bridge closed. Replacement necessary.

26.A.4.8 Substructure Condition Rating

Code	Description
9	EXCELLENT CONDITION. No noticeable or noteworthy deficiencies that affect the condition of the superstructure. Insignificant scrape marks caused by drift or collision.
8	VERY GOOD CONDITION. Shrinkage cracks, light scaling, or insignificant spalling that does not expose reinforcing steel. Insignificant damage caused by drift or collision with no misalignment and not requiring corrective action.
7	GOOD CONDITION. Minor cracking with possible leaching or spalls on concrete or masonry unit with no detrimental effect on bearing area. Leakage of expansion devices have initiated minor cracking. Some rusting of steel without measurable section loss. Insignificant decay, cracking, or splitting of timber. Minor scouring may have occurred.
6	SATISFACTORY CONDITION. Minor deterioration or disintegration, spalls, cracking, and leaching on concrete or masonry units with little or no loss of bearing area. Corrosion of steel section, but no measurable section loss. Some initial decay, cracking, or splitting of timber. Fire damage limited to surface scorching of timber with no measurable section loss. Shallow, local scouring may have occurred near foundation.
5	FAIR CONDITION. Concrete or masonry units may exhibit some section loss with exposed reinforcing steel possible. Measurable, but minor section loss in steel members. Moderate decay, cracking, or splitting of timber; a few secondary members may need replacement. Fire damage limited to surface charring of timber with minor, measurable section loss. Some exposure of timber piles as a result of erosion, reducing the penetration. Scour may be progressive and/or is becoming more prominent with a possibility of exposing top of footing, but no misalignment or settlement noted.
4	POOR CONDITION. Structural cracks and advanced deterioration in concrete and masonry units. Extensive section loss in steel members. Substantial decay, cracking, splitting, or crushing of primary timber members, requiring some replacement. Fire damage with significant section loss of timber that may reduce the load carrying capacity of the member. Extensive exposure of timber piles as a result of erosion, reducing the penetration and affecting the stability of the unit. Additional cross bracing or backfilling is required. Extensive scouring or undermining of footing affecting the stability of the unit and requiring corrective action.
3	SERIOUS CONDITION. Severe disintegration of concrete. Generally, reinforcing steel exposed with advanced stages of corrosion. Severe section loss in critical stress areas. Major fire damage to timber that will substantially reduce the load carrying capacity of the member. Bearing areas seriously deteriorated with considerable loss of bearing. Severe scouring or undermining of footings affecting the stability of the unit. Settlement of the substructure may have occurred. Shoring considered necessary (not just precautionary) to maintain the safety and alignment of the structure.
2	CRITICAL CONDITION. Concrete cap is soft and spalling with reinforcing steel exposed with no bond to the concrete. Top of concrete cap is split or concrete column has undergone shear failure. Structural steel members have critical section loss with holes in the web and/or knife-edge flanges typical. Primary timber members crushed or split and ineffective. Scour is sufficient that substructure is near state of collapse. Pier has settled.
1	"IMMINENT" FAILURE CONDITION. Bridge closed. Corrective action may put back in light service.
0	FAILED CONDITION. Bridge closed. Replacement necessary.

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Appendix 26.B
Bridge Inspection Safety Task Analysis

1. **Preparation.** Review related tasks and safety programs in the *DOT&PF Safety Manual*:

Tasks	Programs (*Manual* Chapter)
Ladders	3.1
Personal Protective Equipment	8.1
Fall Protection	8.3
Confined Space Entry	9.3
Hand Tools (Power and Manual)	14.1

2. **General.**
   - Each inspection team will consist of at least two individuals.
   - Each inspection team will have a first-aid kit readily available.
   - Each inspection team will have capability to notify emergency services by radio, cell phone, or satellite phone.

3. **Traffic Control.** Determine the appropriate traffic control requirements:
   - Where possible, schedule bridge inspections at low traffic periods.
   - On heavily traveled routes and where blind corners lead into a bridge, consider using advance warning signs to alert the traveling public of bridge inspection activities. Place signs as shown in the approved traffic control plans.
   - When approaching a bridge or when parked on the roadside, consider using emergency flashers and strobe light or flashing bars on signal boards.
   - Where possible, to alert drivers of inspection activities, park inspection vehicles behind the traffic rail or off the shoulder as far as possible, yet in a location that maximizes visibility of emergency flashers and strobe light or flashing bars on signal boards.
   - As needed, use traffic cones to alert drivers while inspecting in the shoulder area. Do not place traffic cones inside the fog line.
   - When a lane closure is required, provide traffic control in accordance with an approved traffic control plan.

4. **Potential Hazards.** While inspecting the bridge, be aware of the following potential hazards:
   - Moving traffic.
   - Loose riprap and embankment materials, steep embankments and embankment drop-off, tripping hazards.
   - Overhead hazards including bridge rail posts, girder flanges, drains, and protruding nails.
   - Potential cutting hazards including trash, debris, and exposed metal edges.
   - Locate all utilities on the bridge prior to inspecting. Use caution if there is an observed break in the conduit or protective sheathing.

- Potential insect and animal hazards. Minimize disturbance of bird droppings to avoid exposure to histoplasmosis.

5. **Protective Equipment.** Wear appropriate personal protective equipment consistent with the hazard:

   - Fluorescent vests are required equipment at all times.
   - Fluorescent pants are required equipment during nighttime hours (sunset to sunrise).
   - Use proper fall protection equipment when inspecting from under the bridge unit or aerial platform, and when performing similar activities at heights greater than 6 feet above the ground.
   - Wear hard hats when inspecting from under the bridge unit or aerial platform and around potential head injury hazards.

6. **Confined Spaces.** Adhere to the following procedures when entering confined spaces:

   - An attendant must be present outside the confined space and must be able to communicate with the inspectors using a two-way radio.
   - Follow established lock-out/tag-out procedures.
   - Prior to entering a confined space, test the inside air using a calibrated 4 gas air monitor.
   - If the air is acceptable, then enter keeping the monitor with you. Note air monitor readings and times in report.
   - If the air monitor readings are not acceptable, then do not enter the confined space.
   - Bridges with box-type superstructures are considered a confined space.
   - Deep abutments with limited access are considered a confined space.
   - A culvert is a confined space if one or more of the following are present:
     - Contains or has the potential to contain a hazardous atmosphere.
     - Material blocking safe access through the structure.
     - An internal configuration such that the inspector cannot see from one end to the other.
     - Water depth and/or water current pose a potential hazard.

7. **Culvert Hazards.** Assess culverts for hazards prior to entering. Proceed only if safe. Check based on the confined space criteria.

8. **Stream Cross Sections.** Adhere to the following while taking stream cross sections from the bridge deck:

   - When deemed necessary, while one inspector is taking soundings, the other inspector will be on the bridge deck assisting and acting as a traffic spotter.
   - Do not expose the upper torso out beyond the vertical plane of the railing.
   - Lower the body’s center of gravity by kneeling or crouching

9. **Wading and Probing Inspection.** Adhere to the following while performing wading and probing inspections:

   - Prior to entering the water, determine the water depth visually or by measuring from the bridge deck.

- Prior to entering the water, notify another inspection team member. When deemed necessary, wear a life jacket and have the other inspection team member remain in close proximity and continuous visual contact with the person wading.

- Be sure that there is firm footing and a sound bottom. Probe areas as necessary.

- Consider water depth, velocity, and debris present while assessing conditions.

- To avoid becoming stuck, use extreme caution while walking and wading on silty stream banks and tidal mud flats.

10. **Ladder Use.** Review the manufacturer's recommended guidelines for the proper use of ladders.

11. **Traffic Spotter.** When deemed necessary, while one inspector is chain dragging the concrete bridge deck, the other inspector will be on the bridge deck assisting and acting as a traffic spotter.

12. **Hot and Cold Conditions.** Take extra precautions to prevent heat and cold stress when working in hot or cold temperatures.

13. **Removing Paint.** Wear approved respirator and eye protection when removing paint from steel members by chipping, scraping, and wire wheel:

    - Avoid inhaling or ingesting paint debris.
    - Brush paint debris off clothes and wash hands prior to eating.

14. **Vagrants.** Notify local authorities of vagrants under bridges. Inspect the bridge after the individual has been removed or has left the area.

15. **Accident Reports.** Report accidents as directed in Chapter 2.9 of the *DOT&PF Safety Manual*:

    - Immediately contact law enforcement of a vehicle accident with injury or damage exceeding $2,000. Notify the supervisor and regional Safety Officer as soon as possible.

    - Contact OSHA within eight hours of an occupational fatality or hospitalization.

    - Immediately notify the supervisor of employee accidents.

    - Notify Risk Management of property damage accidents.

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Appendix 26.C
Quality Control/Quality Assurance

26.C.1 Inspection Quality Control (QC) Procedures

The following applies:

1. **General Inspection Requirements**

    - All inspection team leaders must meet 23 CFR 650.309 team leader requirements. The Program Manager maintains a list of qualified team leaders performing bridge inspections for the state of Alaska.

2. **Inspection Frequency**

    Routine, fracture critical member, special and underwater bridge inspections

    - Complete routine, fracture critical member, special, and underwater bridge inspections within the identified calendar month. Within 30 days of the inspection team's return to the office, notify the FHWA Alaska Division, Structural Engineer of all occurrences where inspections are not completed within the identified calendar month. Include the following information:
        - bridge number and name,
        - reason the bridge could not be inspected, and
        - proposed actions to complete the inspection.

3. **Inspection Findings Which May Affect Load Capacity**

    For all structures with an increase in dead load (typically a change in asphalt wearing surface thickness greater than 1 inch) or a decrease in the deck, superstructure, or substructure NBI condition rating to 4 (poor) or lower, the inspector shall:

    - In consultation with the Bridge Inspection Manager, determine if the structure's load rating requires updating and then send an email to the Program Manager and Bridge Inspection Manager discussing the situation.

    - As necessary, load rate the structure to reflect current conditions and provide the load rating to the Bridge Inspection Manager within 60 days of the consultation.

4. **In-House Inspections and Reports**

    - Perform independent review of all inspection reports for quality, spelling, completeness, photograph labeling, and consistency.

    - Inspectors must provide the Program Manager written justification when:
        - changing an NBI condition rating by two or more,
        - lowering an NBI condition rating to 4 (poor) or less
        - raising an NBI condition rating from a 4 (poor) or less

5. **Consultant Inspections and Reports**

    - Perform detailed review of all draft consultant reports for quality, spelling, completeness, photograph labeling, and consistency and provide written comments to consultant. Ensure comments are addressed prior to final report printing.

26.C.2 Inspection Quality Assurance (QA) Procedures

The following applies:

1. **In-House Inspections and Reports**

   - Program Manager shall annually generate a list of bridges on which NBI condition ratings have changed by two or more or where an NBI condition rating is lowered to 4 (poor) or less, and verify that inspectors have provided written justification for these actions.
   - Program Manager shall annually generate a list of bridges on which deck, superstructure, or substructure NBI condition ratings have decreased to 4 (poor) or less, and verify that the need to update load ratings has been considered.
   - Program Manager shall perform independent review of at least three inspection reports from each routine inspection team and one inspection report from each special inspection team.
   - Program Manager or a designated representative shall annually perform independent field reviews of routine bridge inspections completed by two inspection teams. The field reviews will independently assess the following items on 5% to 10% of the bridges inspected by those inspection teams:
     - NBI condition ratings,
     - element condition ratings,
     - inspection observations,
     - work candidates, and
     - overall report completeness including signing, hydraulic sheet, and photographs.

2. **Consultant Inspections and Reports**

   - Program Manager evaluates significant findings and recommendations and ensures that information is entered into BrM. This information shall clearly identify from which inspection the findings and recommendations are made and who entered the information into BrM using the format in the following example:

     The following comments are from previous fracture critical member inspections:

     1) *The upper lateral bracing rods between U2 and U2' are loose.*
     2) *US L1-U1 has significant impact damage.*
     3) *Other verticals and diagonals have minor damage.*
     4) *Tack welds typical on truss members.*

     For additional information, refer to MM/DD/YYYY consultant inspection report. Entered by ABC.

   - Program Manager or a designated representative shall make periodic field site visits to observe consultant bridge inspections. The Program Manager will observe at least one bridge inspection performed by each consultant under contract with a maximum frequency of four years, with two years being the recommended frequency. The field site visits will focus on verifying that contract requirements are met including:
     - traffic control is implemented in accordance with the approved TCP,
     - approved Engineer-in-Charge and inspectors are on-site completing the inspection,
     - required bridge members are inspected, and
     - significant findings including those not accessible by the Program Manager are discussed.

27. Load Rating and Posting

27.1. Load Rating

Load rating bridges is an important function of the DOT&PF Bridge Section. It allows the evaluation of existing structures in a comparative manner.

Bridges designed to former standards are compared to contemporary standards, in conjunction with an appraisal of the bridge condition, through the Inventory Rating. The Operating Rating is used to evaluate permit overloads on structures.

FHWA requires the evaluation of the Inventory Rating and the Operating Rating for every bridge structure. Bridge owners report these values to FHWA annually when reporting other required NBI data.

27.1.1. Definitions

§650.305 of the NBIS defines load rating as:

"The determination of the live load carrying capacity of a bridge using bridge plans and supplemented by information gathered from a field inspection."

In addition, the following definitions apply:

Inventory Rating: The load level that can safely use an existing structure for an indefinite period of time.

Operating Rating: The maximum permissible load level to which the structure may be subjected for the load configuration used in the rating.

27.1.2. Responsibilities

The Bridge Section is responsible for determining the load-carrying capacities of all non-federal publicly owned and maintained bridges in the state of Alaska that are open to the public. The DOT&PF procedures and methodology as presented in Section 27.1 meet all NBIS requirements.

Bridge inspectors must load rate those bridges that have noted any physical changes from the previous inspections that affect structural capacity. In addition, the bridge engineer must load rate all new designs. The bridge engineer submits this load rating to the Bridge Management Unit after approval of shop drawings and prior to opening to traffic.

The Bridge Section determines the need for load posting of all state-owned bridges. The Chief Bridge Engineer orders that such bridges be posted. For non-state bridges, the DOT&PF recommends to the owner the appropriate load posting for bridges under its jurisdiction.

Load rating files, including calculations, are located in separate files in the Bridge Management Unit. Where calculation files are large, the cover sheet is copied and placed in the load rating file with instructions indicating where the complete report may be found.

Load posting recommendations are included in the load rating files.

27.1.3. Methodology

There are three methods for bridge evaluation included in the AASHTO *Manual for Bridge Evaluation* (MBE):

1. Allowable Stress Rating (ASR),
2. Load Factor Rating (LFR), and
3. Load and Resistance Factor Rating (LRFR).

Load rate steel and concrete bridges using the LFR method and timber bridges using the ASR method. In addition, load rate new and replacement bridges designed with the HL-93 load using the LRFR method.

DOT&PF policy is that all critical bridge members be load rated (e.g., decks, girders, floorbeams, stringers, hangers, damaged members, gusset plates, culverts) and that load ratings include values for moment, shear and, where applicable, axial stresses.

The rater must specify live load type, placement for maximum stress, distribution, and impact. Include the following cases for all LFR load ratings:

- inventory with multiple lanes and impact included
- operating with multiple lanes and impact not included
- operating with one lane centered on the bridge and impact not included

27.1.4. Thresholds for Re-rating Existing Bridges

Bridges must be re-load rated when a bridge inspection reveals a quantifiable change in the bridge condition (e.g., increased metal section loss) or change in loading (e.g., change in wearing surface thickness greater than 1 inch, addition of a utility greater than 12 inches in diameter).

The Bridge Management Unit is responsible to ensure bridges are re-load rated but, in most cases, the inspectors complete the load rating in consultation with the Bridge Management Unit.

27.1.5. Load Rating Practices

DOT&PF standardized procedures regarding LRFR load ratings are still under development. For interim procedural recommendations, contact the Bridge Management Unit.

DOT&PF has adopted the following practices for ASR and LFR load ratings of bridges in conjunction with the *AASHTO Standard Specifications for Highway Bridges 17th Edition*, unless otherwise noted:

1. Evaluate both interior and exterior girders.
2. Do not consider deflection in the load rating of structures.
3. Concrete stresses will be as per the as-built plans and specifications. The rater may increase the 28-day strength value of cast-in-place members by 25 percent to account for aging when the in-situ concrete is a minimum of five years old and all evidence shows it is completely sound.
4. In cases where the LFR cast-in-place deck rating requires posting per Section 27.2.1, calculate the moment live load with the LRFD Specifications equivalent strip method (Articles 4.6.2.1.3 and 4.6.2.1.6). In the event that the LRFR live load moment is less than the LFR live load moment, reevaluate the LFR load rating with the revised live load. Report a separate load rating for each live load method.
5. Use the lever rule to calculate the distribution factor if the lever rule value is less than the distribution factor from Table 3.23.1 or “S/D.”
6. Where a pedestrian walkway is separated by a traffic barrier, add an additional lane of traffic when determining “D” in Article 3.24.3.
7. In prestressed girders, check for the minimum shear value from h/2 to midspan.
8. The stress in the prestressing strands shall be linearly varied from 0 ksi to f_{pe} in a distance of 50 diameters from the end of girder (25 inches for ½-inch diameter strands). If this transfer length extends past h/2 from the support, it must be included in the load rating.
9. Use the stress in prestressing strands after all losses shown on the plans. Assume 45 ksi for all losses if no value exists and the bridge was designed to or prior to the *AASHTO Standard Specification for Highway Bridges 17th Edition.* Otherwise, calculate losses according to Section 14.4.2.
10. Increase the live load shear in girders according to Section 13.1.2.
11. When calculating either the Inventory or Operating Rating for bulb-tee girders, use the distribution factor “D” in the concrete beam formula in Article 3.23.4. Determine J utilizing the equation for stocky open sections provided in the current *LRFD Specifications*. Historically, K = 2.2 has been used for load rating as the equation for J was not adequately defined. When an LFR load rating is updated per Section 27.1.4, revise the K value.
12. For prestressed girders, use the average stirrup spacing for shear ratings. See Section 6A.5.8 of the current *Bridge Manual for Evaluation* for guidance where d_v = d and θ = 45 degrees.
13. In cases where the LFR shear load rating is governed by the term $V_s = 8\sqrt{f'_c}b_w d$ in Article 9.20.3.1 of the *AASHTO Standard Specification for Highway Bridges 17th Edition*, substitute the *LRFD Specifications* term $V_n = 0.25f'_c b_v d_v \underline{+V_p}$ (LRFD Eq. 5.~~8~~7.3.3-2) as the limiting equation.
14. For prestressed girders, use a maximum effective flange width of 84 inches for calculating section properties and the full flange width for dead load calculations.
15. When utilizing Equation 10-129c for composite sections, use the following equations consistent with the first yield of the *LRFD Specifications.*

$$M_u = \frac{5M_p - 0.85F_y S_{x,n} R}{4} + \frac{0.85F_y S_{x,n} R - M_p}{4}\left(\frac{D_p}{D'}\right) \geq M_{y1}$$

where:

$$M_{y1} = S_{x,n}\left[RF_y - \frac{1.3M_{DC1}}{S_{x,steel}} - \frac{1.3M_{DC2}}{S_{x,3n}}\right] + 1.3M_{DC1} + 1.3M_{DC2}$$

$S_{x,steel}$ = steel only section modulus
$S_{x,n}$ = short-term composite section modulus
$S_{x,3n}$ = long-term composite section modulus

16. In composite bridges, calculate the effective flange width as the minimum of the following:

    a. Interior Girder, $(b_{eff})_{interior}$

       1) girder spacing
       2) span length / 4
       3) 12 * slab thickness + 0.5 * top flange width

    b. Exterior Girder, $(b_{eff})_{exterior}$

       1) 0.5 * $(b_{eff})_{interior}$ + overhang width
       2) 0.5 * $(b_{eff})_{interior}$ + span length / 8
       3) 0.5 * $(b_{eff})_{interior}$ + 6 * slab thickness + 0.25 * top flange width

    For further information see Article 4.6.2.6 of the 2006 Interim Revisions of the *AASHTO LRFD Bridge Design Specifications*, 3rd Edition.

17. Evaluate log bridges in accordance with the publication “Design Guide for Native Log Stringer Bridges,” USDA, Forest Service, Region 10, by Frank Muchmore.

18. When load rating timber decks, use wet-use stress in all cases. Evaluate other timber members with wet-use stresses in the marine and transitional climatic zones. Structures in the continental and arctic climatic zones may use dry-use stresses. Review bridge inspection reports for specifics on the moisture condition of individual members.

19. If the grade and species of a sawn timber member is unknown, assume Douglas Fir No. 2 for that member’s strength properties unless the member is creosoted. For creosoted timber assume Douglas Fir No. 1 or Douglas Fir No. 1 & Btr.

20. Following the *AASHTO Manual for Bridge Evaluation* Articles 6A.2.3.4 and 6B.6.2.4, do not include pedestrian loading simultaneous with vehicular loads. Where significant pedestrian loading is expected to coincide with the maximum vehicular loading, consult with the Chief Bridge Engineer, Bridge Management Engineer, or Load Rating Manager prior to including pedestrian loading. Do not include pedestrian loading in one lane centered operating ratings discussed in Section 27.1.3.

27.1.6. Dimensions

Use the dimensions as shown in the shop drawings, as-built drawings, and construction drawings unless field-measured dimensions deviate significantly from the plan dimensions or there are no plans.

The rater may reduce the structural section properties of a deteriorated component based on field measurements and engineering judgment as derived from an inspection of the bridge.

27.1.7. Load Rating Quality Control (QC) Procedures

Perform load ratings in accordance with the AASHTO *Manual for Bridge Evaluation* and Section 27.1.

All load ratings will be independently reviewed for reasonableness and general conformity with the state’s load rating practices. On the Load Rating Summary Sheet, the reviewer will note “Reviewed for Conformity” and sign each load rating reviewed.

A minimum of 10 percent of all bridges load rated in a calendar year will receive an independent load rating check, using independently developed assumptions.

A 10 percent difference in the calculated live load, dead load, nominal capacity, and equivalent HS values is considered acceptable. Reconcile discrepancies to meet these requirements. The more conservative equivalent HS value is reported.

Axle Group	Total Group Weight (kips) [1]	Minimum Spacing (ft)
Single	20.0	-
Tandem	38.0	3'-6"
Triple – Option 1	42.0	3'-6"
Triple – Option 2	43.5	5'-0"
Triple – Option 3	45.0	6'-0"
Quad	50.0	3'-6"

[1]*Distribute weights evenly to each axle*.

Table 27-1
Posting Axle Groups

27.2. Load Posting

27.2.1. Thresholds for Posting Existing Bridges

When the LFR Operating Rating is equal to or less than 3 tons, then the bridge owner must close the bridge.

The rater must evaluate a bridge for posting when a concrete deck has a LFR Inventory Rating less than HS12.5 (RF < 0.625).

The rater must evaluate a bridge for posting when a timber deck or an orthotropic steel deck has a LFR Inventory Rating less than HS15 (RF < 0.75).

The rater must evaluate a bridge for posting when a critical member, other than the deck, has a LFR Inventory Rating less than HS15 (RF < 0.75).

27.2.2. Loads for Posting

The following applies:

1. **Decks.** A single axle controls. The posting should be for the maximum axle weight that the deck can carry at its LFR Inventory Rating. Base posting on a 20-inch tire width.

2. **Stringers, Girders, Floorbeams, etc.** Evaluate for probable legal load configurations consisting of a single, tandem, triple, and quad axle groups (See Table 27-1). In addition evaluate the posting loads in the current *Manual for Bridge Evaluation*. Post the structure for each axle and vehicle configuration that exceeds the critical-member capacity based on the LFR Inventory Rating.

27.2.3. Load Posting Notices

The Bridge Section sends posting notices to bridge owners and requests photographic evidence that the posting has been implemented. The correspondence requests that postings be implemented within 30 days or justification for not posting within this time frame.

Load posting notices for DOT&PF-owned structures are sent to the Region Director with copies to the Region Maintenance Chief, Maintenance Superintendent, Division Director MS&CVE, Chief MS&CVE, Administrative Supervisor of the Commercial Vehicle Customer Support Center MS&CVE, and the FHWA Alaska Division Structures Research Engineer.

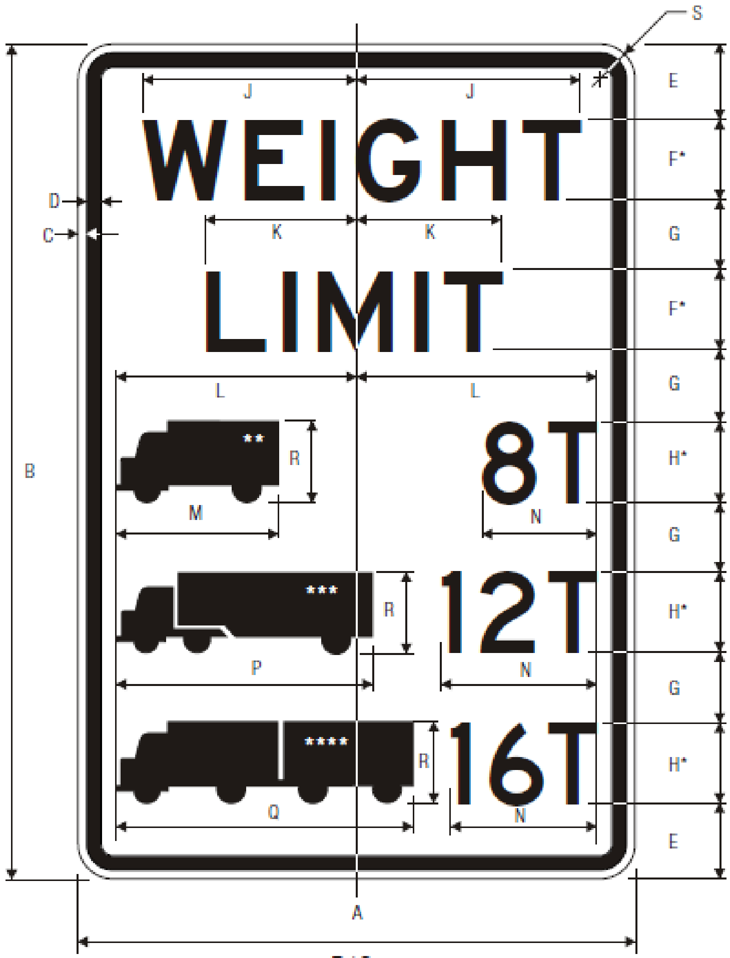
Load posting notices for other state agency-owned and local agency-owned structures are sent to the identified owner with a copy to the DOT&PF Region Director, Maintenance Chief, Maintenance Superintendent, Division Director MS&CVE, Chief MS&CVE, Administrative Supervisor of the Commercial Vehicle Customer Support Center MS&CVE, and the FHWA Alaska Division Structures Research Engineer.

Load posting notices sent to the owners are filed in the hard copy bridge folder, e-vault, and load rating files.

The Bridge Section maintains a suspense file mechanism to track and send bi-monthly follow-up correspondence to bridge owners notifying them of the need to load post the bridge.

27.2.4. Signing

Use only those signs approved for use in the *Alaska Sign Design Specifications* (*ASDS*) for posting. For simple understanding of load posting signs, where possible, post using gross weight limits only (example shown in Figure 27-2). The following figures duplicate the applicable signs from the *ASDS*.



R12-5
WEIGHT LIMIT

*Series 2000 Standard Alphabets.
**See page 6-4 for design
***See page 6-5 for design
****See page 6-6 for design

	A	B	C	D	E	F	G	H	J	K	L	M
C	24	36	.375	.625	3.25	3.5 E	3	3.5 D	9.465	6.403	10.375	7
	30	42	.5	.75	3.75	4.5 E	3	4.5 D	12.016	8.192	13.188	9
	36	48	.625	.875	5	5 E	3.25	5 D	13.350	9.099	15	10
	48	60	.75	1.25	6	6 E	4.5	6 D	16.02	10.918	19	12

N	P	Q	R	S
VAR	11	12.813	3.5	1.5
VAR	13.438	16.438	4.5	1.875
VAR	15.438	18.375	5	2.25
VAR	19.688	22	6	3

COLORS: LEGEND — BLACK
BACKGROUND— WHITE (RETROREFLECTIVE)

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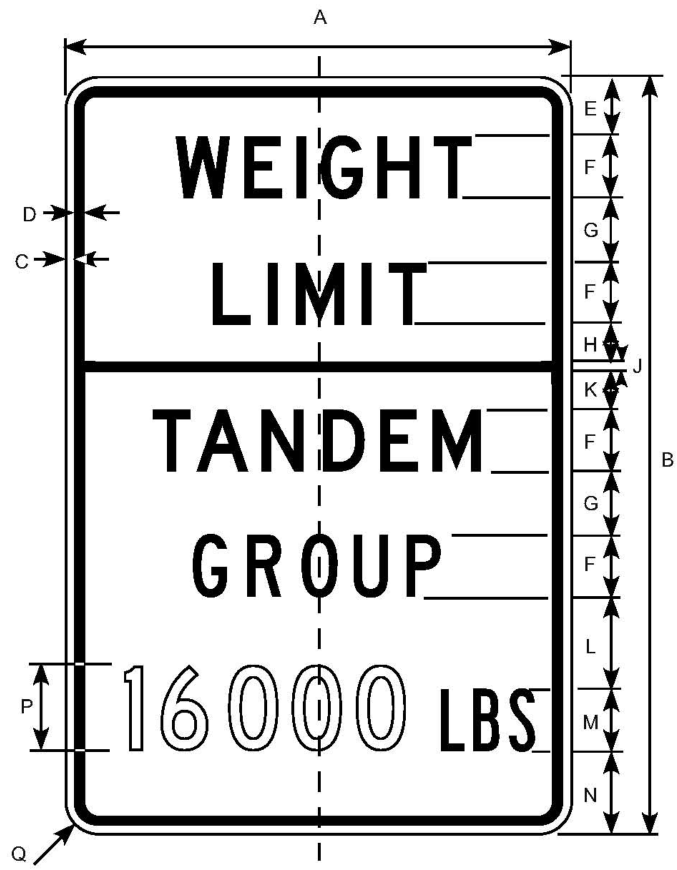
Figure 27-2
ASDS Signs Example
(Page 1 of 5)

R12-100



Border and Legend: Black
Background: White

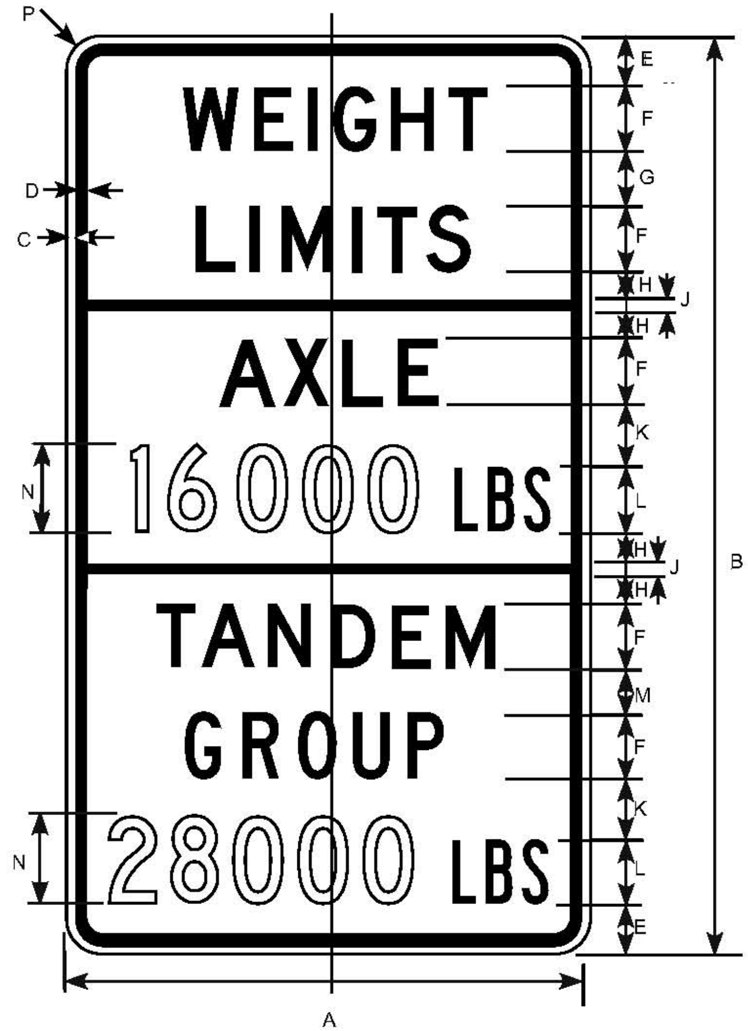
Prior to the 2002 publication of this manual,
this sign was numbered R12-5A.

Road	Dimensions (inches)														
Class	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**J**	**K**	**L**	**M**	**N**	**P**	**Q**
Conv & Min	24	36	0.38	0.63	2.75	3C	3	1.69	0.63	1.69	4.13	3B	4.13	4C	1.5

ALASKA SIGN DESIGN SPECIFICATIONS 4/8/02

Figure 27-2
ASDS Signs Example
(Page 2 of 5)

R12-101



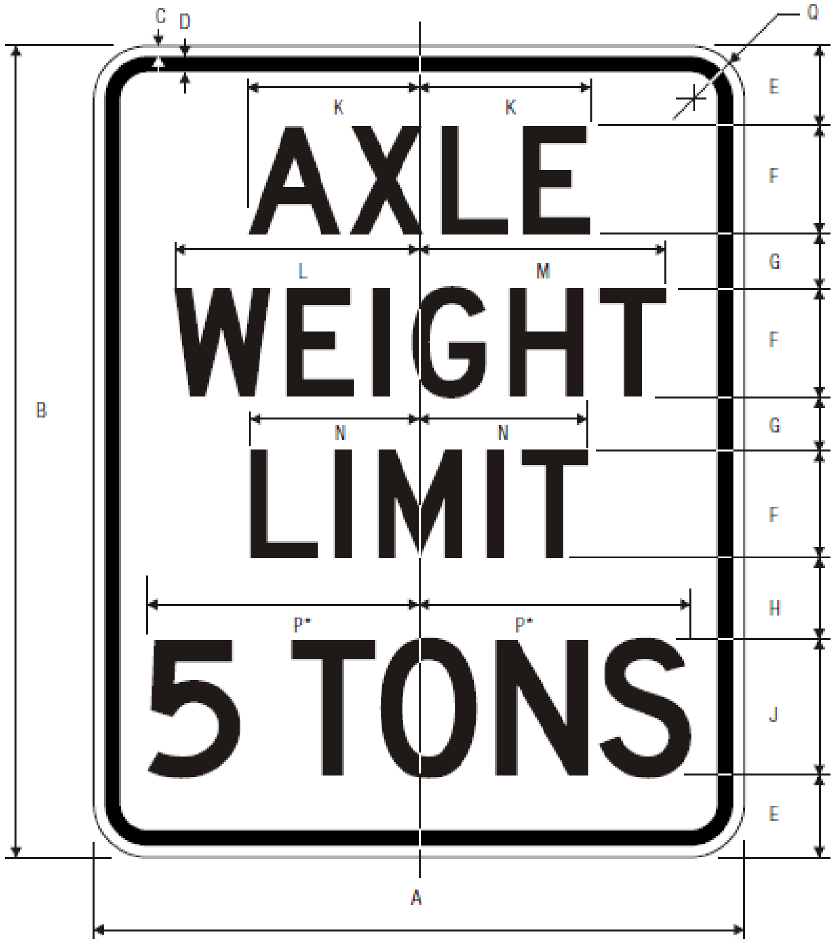
Border and Legend: Black
Background: White

Prior to the 2002 publication of this manual, this sign was numbered R12-5B.

Road	Dimensions (inches)													
Class	A	B	C	D	E	F	G	H	J	K	L	M	N	P
Conv & Min	24	42	0.38	0.63	2.25	3C	2.50	1.25	0.63	2.88	3B	2	4C	1.5

ALASKA SIGN DESIGN SPECIFICATIONS 4/8/02

Figure 27-2
ASDS Signs Example
(Page 3 of 5)



R12-2

AXLE WEIGHT LIMIT

*Reduce spacing 50%.

	A	B	C	D	E	F	G	H	J	K	L
C	24	30	.375	.625	3.25	4 D	2	2.5	5 D	6.875	9
	36	48	.625	.875	5.25	6 D	3.5	4.5	8 D	10.313	13.5

M	N	P	Q
9.5	6.313	10	1.5
14.25	9.438	15	2.25

COLORS: LEGEND — BLACK
BACKGROUND— WHITE (RETROREFLECTIVE)

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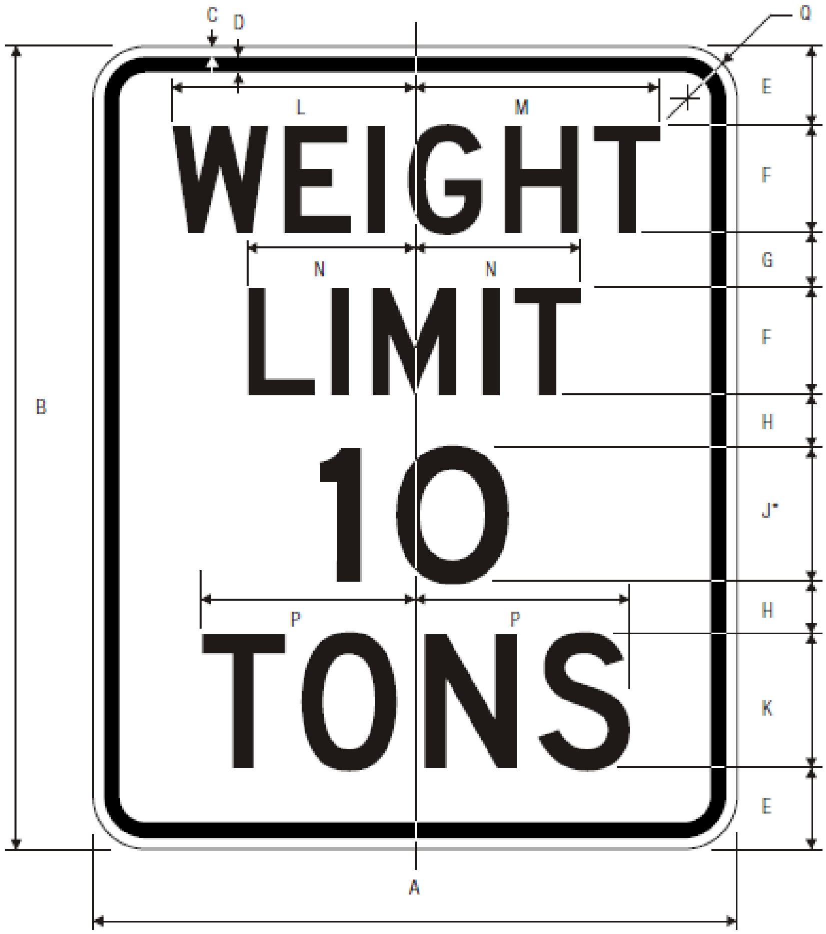
Figure 27-2
ASDS Signs Example
(Page 4 of 5)



R12-1

WEIGHT LIMIT

*Optically space numerals about centerline.

	A	B	C	D	E	F	G	H	J	K	L
C	24	30	.375	.625	3	4 D	1.75	2.125	5 E	5D	9
	36	48	.625	.875	4.75	6 D	3	3.75	8 E	8 D	13.5

M	N	P	Q
9.5	6.313	8.25	1.5
14.25	9.438	13.25	2.25

COLORS: LEGEND — BLACK
BACKGROUND— WHITE (RETROREFLECTIVE)

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Figure 27-2
ASDS Signs Example
(Page 5 of 5)

28. Bridge Management

28.1. Responsibilities

This chapter briefly discusses the responsibilities of DOT&PF units that are involved with managing Alaska's bridges.

28.1.1. Bridge Section

The Bridge Management Unit is responsible for the state's bridge management activities. These include:

- collecting technical data during inspections. The state inspects all bridges on public roads except for those that are federally owned;
- developing and distributing inspection reports;
- developing program work recommendations, which are provided to the regions and local agency bridge owners;
- reporting on bridge performance measures as part of the state Office of Management and Budget's "Key Performance Indicators" for DOT&PF;
- responding to internal and external bridge data inquiries;
- developing and prioritizing bridge rehabilitation and replacement lists to decision-makers;
- identifying and assisting the regions with programming bridge rehabilitation projects;
- load rating bridges; and
- assisting Measurement Standards and Commercial Vehicle Enforcement with evaluating overweight permit requests.

28.1.2. Program Development

The Division of Program Development uses the information provided by the Bridge Section to include bridge projects in the Statewide Transportation Improvement Plan (STIP).

28.1.3. Regional Offices

Department regional offices use the information provided by the Bridge Section to help develop their proposed overall program of capital-improvement projects for DOT&PF funding.

28.1.4. Maintenance Division

The DOT&PF Maintenance & Operations (M&O) electronically extracts work candidates from the AASHTOWare Bridge Management (BrM) database for use in the Statewide Maintenance Management System.

28.2. BrM Software

AASHTOWare Bridge Management (BrM) is an AASHTO bridge management software package that relies upon collected condition data and cost data for bridge elements (e.g., girders, piers, railings). DOT&PF administers and maintains the BrM database in the Oracle environment

State DOTs may use this data to identify least-cost (optimal), long-term preservation and improvement policies for a network of bridges.

DOT&PF currently uses BrM to warehouse the state's NBI data and to collect and store all element-level bridge inspection data.

BrM stores inventory and inspection information on bridges in a relational database that supports modeling, analysis, and reporting tools to facilitate project, budget, and program development. BrM assists in the formulation of network-wide preservation and improvement policies for use in evaluating the needs of each structure in the network, and makes project recommendations for DOT&PF program of capital projects. BrM analyzes the impact of various project alternatives on the performance of individual structures or a network of structures.

28.2.1. Bridge Management Process

The bridge management process begins with the building of a relational database that includes importing NBI data and adding element-level inspection information.

DOT&PF uses this information to develop prioritized lists that are provided to the Division of Program Development and the regions for use in preparing project scopes. A brief discussion of the prioritization model is included in Appendix 28.A.

28.2.2. Elements

In its use of element-level inspection data, BrM subdivides the main components of a typical bridge (e.g., deck, superstructure, substructure) into numerous elements to add more detail and precision.

DOT&PF's bridges can be defined from a set of National Bridge Elements (NBEs), Bridge Management Elements (BMEs), and (Agency-Developed Elements (ADEs), as defined by the AASHTO *Manual for Bridge Element Inspection*.

28.2.3. Bridge Inspection

Chapter 26 discusses the Alaska Bridge Inspection Program.

28.3. Federal-Aid Program

Federal Highway Administration (FHWA) funds are eligible for use for replacement, rehabilitation, seismic retrofit, preservation, scour countermeasures and application of anti-icing or deicing compositions to highway bridges on and off federal-aid highways.

28.3.1. Sufficiency Rating/Structural Deficiency/Functional Obsolescence

The sufficiency rating (SR) is based on a 0 to 100 scale (100 being best), and is calculated using a formula that incorporates four factors:

- structural adequacy and safety (55 percent),
- serviceability (30 percent),
- essentiality for public use (15 percent), and
- special reductions (up to 13 percent).

Structurally Deficient

In general a bridge is categorized as structurally deficient (SD) if the bridge:

1. is in relatively poor condition due to deterioration or damage;
2. has insufficient load-carrying capacity (whether due to the bridge being of older design or due to deterioration); or
3. frequently floods, causing significant traffic delays.

Structurally deficient bridges may require significant maintenance attention, rehabilitation, or replacement. These bridges often require load posting (See Chapter 27).

However, the classification of a bridge as structurally deficient does *not* usually mean that it is in danger of collapse or that it is unsafe.

Functionally Obsolete

In general a bridge is categorized as functionally obsolete (FO) if the bridge:

- is narrow,
- has inadequate under clearances,
- is poorly aligned with the adjacent roadway, and/or
- can no longer adequately service today's traffic,
- occasionally floods, causing significant traffic delays.

Functionally obsolete bridges may not provide the lane widths, shoulder widths, vertical clearances, etc., adequate to serve traffic demand, or the bridge may not be able to handle occasional roadway flooding without causing traffic delays.

By rule, bridges that qualify as both structurally deficient and functionally obsolete are categorized and reported solely as structurally deficient.

For additional coding information, refer to the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, Report No. FHWA-PD-96-001.

28.3.2. MAP-21

National Highway Performance Program

In 2012, the *Moving Ahead for Progress in the 21st Century Act* (MAP-21) became law. MAP-21 consolidated several FHWA funding programs, including the Highway Bridge Program, into the National Highway Performance Program (NHPP).

The cornerstone of MAP-21's highway program transformation is the transition to a performance and outcome based federal-aid program. States will invest resources in projects to achieve individual targets that collectively will make progress toward national goals.

For bridges, the Secretary, in consultation with States, Metropolitan Planning Organizations (MPOs) and other stakeholders, will establish performance measures for bridge conditions. States (and MPOs, where applicable) will set performance targets in support of those measures, and state and metropolitan plans will describe how program and project selection will help achieve the targets.

Performance Based NHS

MAP-21 establishes a performance basis for maintaining and improving the NHS.

States are required to develop a risk and performance based asset management plan for the NHS to improve or preserve asset condition and system performance. The plan development process must be reviewed and recertified at least every four years.

The Secretary will establish performance measures for Interstate and NHS pavements, NHS bridge conditions, and Interstate and NHS system performance. States will establish targets for these measures, to be periodically updated.

MAP-21 requires minimum standards for conditions of Interstate pavements and NHS bridges by requiring states to devote resources to improving their condition until the established minimum is exceeded.

MAP-21 establishes the minimum standards for NHS bridge conditions – if more than 10 percent of the total deck area of NHS bridges in a state is on structurally deficient bridges, the state must devote a portion of NHPP funds to improve conditions.

Surface Transportation Program

MAP-21 continues the STP, providing flexible funding that may be used by states and local governments for projects to preserve or improve conditions and performance on any federal-aid highway, bridge projects on any public roads, facilities for nonmotorized transportation, transit capital projects and public bus terminals and facilities.

Fifty percent of state STP funds must be distributed to areas based on population (suballocated), with the remainder to be used in any area of the state.

Also, a portion of state STP funds (equal to 15 percent of the state FY 2009 Highway Bridge Program apportionment) must be set aside for bridges not on federal-aid highways (off system bridges), unless FHWA determines that the state has insufficient needs to justify this amount.

28.3.3. Prioritization Model

See Appendix 28.A.

Appendix 28.A
Prioritization of Structurally Deficient Bridges

Factors included in the model used to generate prioritized lists of bridges for rehabilitation and/or replacement are:

1. Structural condition - NBI Items 58 (Deck), 59 (Superstructure), 60 (Substructure)
2. Importance (on or off the NHS)
3. ADT
4. Bypass/detour length

Functional obsolescence is not considered in the prioritization model because many narrow bridges are adequate for their ADT. The decision to address functional obsolescence is left to the regions or owners who are most familiar with the use of a bridge. State 3R standards may also require widening when bridges are included within the limits of a larger roadway project.

The load posting status is not considered in the prioritization model because a load posted bridge may be meeting the needs for the level of service it sees and may not require strengthening. The load posting status is provided so that the regions or owners most familiar with the use of a bridge can take this information into account when developing a program. State 3R standards may also require strengthening when bridges are included within the limits of a larger roadway project.

Closed bridges are included in the lists to give a complete accounting of the eligible bridges but not ranked. Closed bridges typically have low ratings but many have been closed with no action taken by the owner to reopen them. The decision to address closed bridges is left to the regions or owners who are most familiar with the needs of the traveling public affected by the closed bridge.

It is possible to look at the data in a variety of ways: All Bridges, All State Owned Bridges, State Owned Bridges On-System, State Owned Bridges Off-System, State DOT Owned Bridges, State DOT Owned-Southcoast Region, State DOT Owned Bridges-Central Region, State DOT Owned-Northern Region, and Non-State Owned Bridges.

Model for Prioritization of Structurally Deficient Bridges

Parameters used and method of calculation is provided below:

Start

If the NBI Deck Rating is ≥ 4, then 1; else

If the NBI Deck Rating is ≤ 3, then 3

Multiplied by:

If the NBI Superstructure Rating is ≥ 5, then 1; else

If the NBI Superstructure Rating is = 4, then 2; else

If the NBI Superstructure Rating is ≤ 3, then 5

Multiplied by:

If the NBI Substructure Rating is ≥ 5, then 1; else

If the NBI Substructure Rating is =4, then 2; else

If the NBI Substructure Rating is ≤ 3, then 5

Multiplied by:

(ADT/5000) ^ (0.25)

Multiplied by:

If on NHS, then 3; else 1

Multiplied by:

If the Detour Length is ≥ 120 miles then 2; else

If the Detour Length is > 50 miles then 1.5; else

If the Detour Length is ≤ 50 miles, then 1

End

Culverts, Pedestrian Bridges, Railroad Bridges, Tunnels and Minor Structures are not included in the prioritized lists.

Additional Guidance on 23 CFR 650 D – Programs – Bridge – FHWA

Highway Bridge Replacement and Rehabilitation Program (23 CFR 650.409)

The National Bridge Inventory will be used for preparing the selection list of bridges both on and off of federal-aid highways. Highway bridges considered structurally deficient or functionally obsolete and with a sufficiency rating of 80 or less will be used for the selection list. Those bridges appearing on the list with a sufficiency rating of less than 50.0 will be eligible for replacement or rehabilitation while those with a sufficiency rating of 80.0 or less will be eligible for rehabilitation. To be considered for the classification of deficient bridge, a structure must be of bridge length, and had not been constructed or had major reconstruction within the past 10 years.

General Qualifications

In order to be considered for either the structurally deficient or functionally obsolete classification, a highway bridge must meet the following:

1. Structurally Deficient
    - A condition rating of 4 or less for:
        - Item 58 – Deck,
        - Item 59 – Superstructures,
        - Item 60 – Substructures, or
        - Item 62 – Culvert and Retaining Walls[1]
    - An appraisal rating of 2 or less for:
        - Item 67 – Structural Condition, or
        - Item 71 – Waterway Adequacy[2]
2. Functionally Obsolete
    - An appraisal rating of 3 or less for:
        - Item 68 – Deck Geometry,
        - Item 69 – Underclearance[3], or
        - Item 72 – Approach Roadway Alignment.
    - An appraisal rating of 3 for:
        - Item 67 – Structural Condition, or
        - Item 71 – Waterway Adequacy[2]

A bridge classified as structurally deficient is excluded from the functionally obsolete category.

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[1] Item 62 applies only if the last digit of Item 43 is coded 19.
[2] Item 71 applies only if the last digit of Item 42 is coded 0, 5, 6, 7, 8, or 9.
[3] Item 69 applies only if the last digit of Item 42 is coded 0, 1, 2, 4, 6, 7, or 8.

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