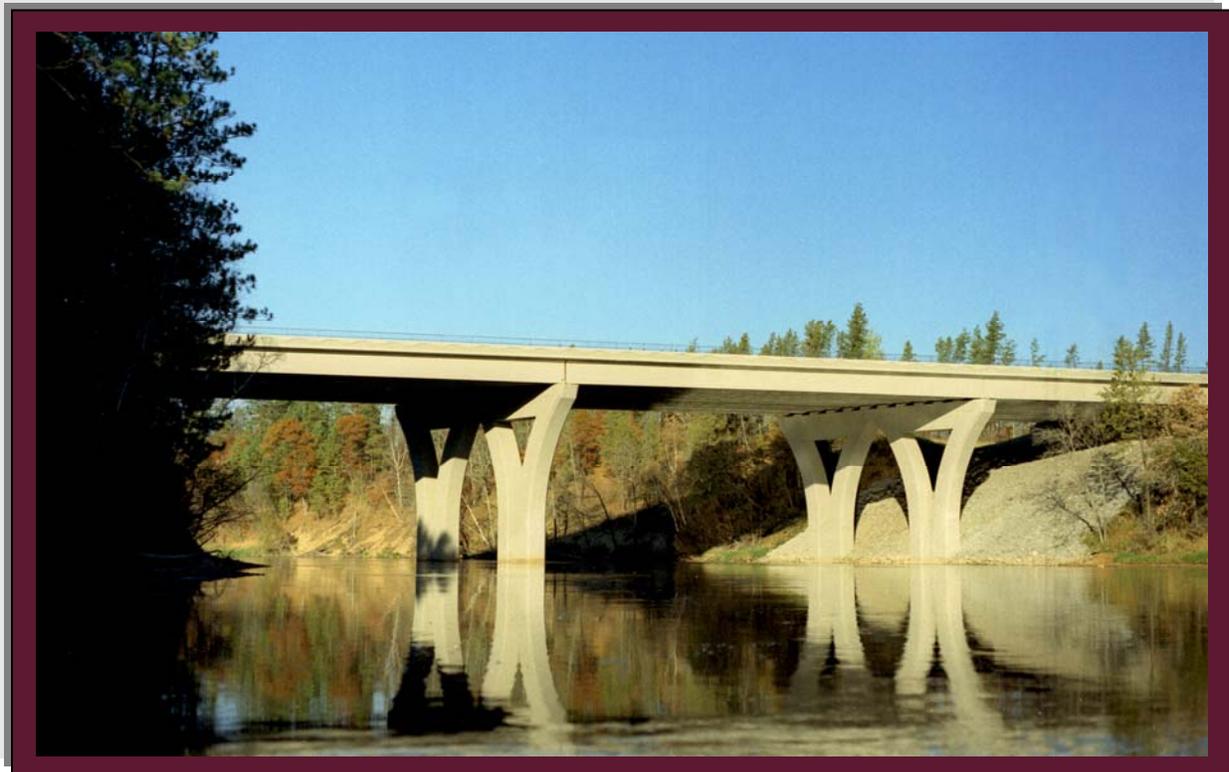


# LRFD Bridge Design Manual



**mn** DEPARTMENT OF  
TRANSPORTATION

BRIDGE OFFICE



MANUAL

5-392

MINNESOTA DEPARTMENT OF TRANSPORTATION

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Bridge Office

# LRFD Bridge Design Manual

MnDOT BRIDGE OFFICE

# LRFD Bridge Design Manual

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JULY 2003   OCTOBER 2003   JANUARY 2004   APRIL 2004   OCTOBER 2004   DECEMBER 2004  
FEBRUARY 2005   MARCH 2005   NOVEMBER 2005   MARCH 2006   APRIL 2006   MAY 2006  
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**APPENDIX A.** Memos to Designers

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#2005-02 REMOVED

#2005-03 REMOVED

#2006-01 REMOVED

#2007-01 REMOVED

#2007-02 Adhesive Anchors Under Sustained Tensile Loads ..... (dated Oct. 3, 2007)

#2007-03 REMOVED

#2008-01 Prestressed Concrete Design – Calculation of Prestress Losses and  
Beam Camber & Deflection.....(dated Sept. 18, 2008)

#2008-02 Truss Bridge Gusset Plate Analysis ..... (dated Oct. 20, 2008)

#2011-01 REMOVED

#2011-02 REMOVED

#2011-03 REMOVED

#2012-01 Discontinued Usage of Plain Elastomeric Bearing Pads and  
Substitution with Cotton-Duck Bearing Pads .....(dated April 12, 2012)

#2012-02 Transition to New  
MnDOT Pile Formula 2012 (MPF12)..... (dated November 21, 2012)

#2013-01 Conversion from Metric to  
U.S. Cust. Rebar Designations .....(dated April 17, 2013)

#2014-01 AASHTO LRFD Article 5.7.3.4 Concrete Crack Control Check  
(dated August 6, 2014)

#2014-02 Inclusion of Informational Quantities in Bridge Plans  
(dated December 23, 2014)

#2015-01 Concrete Mix Design Designations ..... (dated August 10, 2015)

#2016-01 Single Slope Barrier (Type S) Bridge Standards ..... (dated December 09,  
2016)

#2017-01 Edge-of-Deck Thickness on Bridges and Wall Coping Height  
(dated March 28, 2017)

#2017-02 Post-Installed Anchorages for Reinforcing Bars ... (dated October 19, 2017)

#2018-01 New 30MH, 35MH, and 40MH Prestressed Concrete Beams  
(dated December 20, 2018)

#2019-01 Temporary Portable Precast Concrete Barriers on Bridges  
(dated January 30, 2019)

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**1.**  
***INTRODUCTION***

This section contains general information about the manual along with a general description of the Bridge Office and its procedures.

***1.1 Overview of  
Manual 5-392***

This manual contains Mn/DOT Bridge Office policies and procedures for the design, evaluation, and rehabilitation of bridges. Except where noted, the design provisions herein employ the Load and Resistance Factor Design (LRFD) methodology set forth by AASHTO.

Mn/DOT utilizes a decimal numbering system to classify documents. The "5" before the hyphen represents a publication related to engineering. The "300" series of documents is assigned to the Bridge Office; the "90" series indicates that this is a "Manual". The last digit "2" specifies that the subject matter of the document is "Design".

The original bridge design manual, numbered 5-392, provided guidance for the design of highway structures in Minnesota in accordance with allowable stress design methods. Subsequently, it has received periodic updates as design methods have changed. This version of the Bridge Design Manual contains significant changes. It presents Mn/DOT's design practices in conformance with a new design methodology, Load and Resistance Factor Design (LRFD), and also contains fifteen comprehensive design examples.

Use of this manual does not relieve the design engineer of responsibility for the design of a bridge or structural component. Although Bridge Office policy is presented here for numerous situations, content of the manual is not intended to be exhaustive. Therefore, use of this manual must be tempered with sound engineering judgment.

***1.1.1 Material  
Contained in  
Manual 5-392***

After this introductory material, the manual contains material arranged around the following section headings. To simplify locating material, section numbers correspond to those used in the LRFD specifications:

- 1) Introduction
- 2) General Design and Location Features
- 3) Loads and Load Factors
- 4) Structural Analysis and Evaluation
- 5) Concrete Structures
- 6) Steel Structures
- 7) Reserved
- 8) Wood Structures
- 9) Decks and Deck Systems
- 10) Foundations
- 11) Abutments, Piers, and Walls

- 12) Buried Structures
- 13) Railings
- 14) Joints and Bearings
- 15) Ratings  
    Memos

### ***1.1.2 Updates to Manual 5-392***

This manual will be updated multiple times each year as procedures are updated and new information becomes available. Current files for each section of the manual are available on the Bridge Office Web site at: <http://www.dot.state.mn.us/bridge/> .

### ***1.1.3 Format of Manual References***

Each section of the manual contains general information at the start of the section. Design examples (if appropriate) are located at the end of each section. The general content is divided into subsections that are identified with numerical section labels in the left margin. Labels for design example subsections are identified with alphanumeric labels in the left hand margin. The left hand margin also contains references to LRFD Design Specification Articles, Equations, and Tables. These references are enclosed in square brackets.

Within the body of the text, references to other sections of this manual are directly cited (e.g. Section 10.1). References to the LRFD Specifications within the main body of the text contain a prefix of: LRFD.

## ***1.2 General Bridge Information***

A bridge is defined under Minnesota Rule 8810.8000 Subp. 2 as a structure "having an opening measured horizontally along the center of the roadway of ten feet or more between undercopings of abutments, between spring line of arches, or between extreme ends of openings for multiple boxes. Bridge also includes multiple pipes where the clear distance between openings is less than half of the smaller contiguous opening."

In accordance with Minnesota Statute 15.06 Subd. 6, the Commissioner of Transportation has delegated approval authority for State Preliminary Bridge Plans, and State, County and City Final Bridge Plans to the State Bridge Engineer. Plans for all bridge construction or reconstruction projects located on the Trunk Highway System, and plans on County or City highways funded fully or in part by state funds shall be approved by the State Bridge Engineer.

### 1.2.1 Bridge Office

The Bridge Office is responsible for conducting all bridge and structural design activities and for providing direction, advice, and services for all bridge construction and maintenance activities. The responsibilities include:

- Providing overall administrative and technical direction for the office.
- Reviewing and approving all preliminary and final bridge plans prepared by the office and consultants.
- Representing the Department in bridge design, construction and maintenance matters with other agencies.

The Office is under the direction of the State Bridge Engineer. It is composed of sections and units as shown on the organizational chart (Figure 1.2.1.1). Each of these subdivisions with their principal functions is listed as follows:

#### 1) Bridge Design Section

Responsible for the design, plans, and special provisions activities for bridges and miscellaneous transportation structures.

##### a) Design Unit

- i) Designs and drafts bridge design plans for new bridge construction or in-place bridge repairs.
- ii) Reviews bridge plans prepared by consulting engineers.
- iii) Prepares special provisions for bridge plans.
- iv) Designs and drafts plans for miscellaneous transportation structures.
- v) Provides technical assistance, designs, and plans for special bridge and structural problems.
- vi) Assists the Districts and other offices in solving bridge and other structure construction issues.

##### b) Bridge Evaluation Unit

- i) Provides review of fracture critical inspection reports and recommends reevaluation of rating as needed.
- ii) Performs design or rating for special non-bridge structures.
- iii) Analyzes unusual or atypical bridge structures.
- iv) Responds to and prepares plans for repairs and retrofits to bridges damaged by bridge hits.

##### c) State Aid Bridge Unit

- i) Assists local agencies in the planning, designing, and construction of bridge projects.
- ii) Reviews preliminary and final bridge plans for counties, townships, and municipalities within the State of Minnesota which receive State and/or Federal Aid Funds for bridge construction. The bridge plan reviews are conducted to assure they comply with AASHTO LRFD Design Specifications,

- Mn/DOT LRFD Bridge Design Manual, applicable Mn/DOT Technical Memorandums, Mn/DOT Standard Specifications for Construction, applicable Mn/DOT Bridge Special Provisions, and all Mn/DOT policies.
- iii) Serves to assist in the planning and review of miscellaneous structures for local agencies. These structures include, but are not limited to, pedestrian bridges, boardwalks, retaining walls, culverts, parking ramps, park buildings, skyways, and stair towers.
  - iv) Provides technical assistance to local agencies and their consultants in the implementation of new, innovative, efficient and cost effective bridge systems.
  - v) Provides assistance as requested by the local agencies and/or their consultants, with the preparation, setup, and delivery of local bridge training. The training can encompass all aspects of local bridges, such as planning, design, construction, load rating, and inspection.
- d) LRFD Unit
    - i) Maintains LRFD Bridge Design Manual.
    - ii) Provides support to office and consulting engineers concerning LRFD issues.
  - e) Design/Build Unit
    - i) Prepares procurement documents for design/build projects.
    - ii) Provides design oversight for design/build projects.
- 2) Standards, Research, and Automation Section  
Responsible for development of standards and design aids, managing research studies pertaining to bridges, and supporting computing needs in the office.
- a) Bridge Standards Unit
    - i) Provides design aids and standards for the office and for consultants, counties, and cities.
    - ii) Provides oversight for research projects, which involve the Bridge Office.
  - b) Information Resource Management Unit
    - i) Coordinates the development of computer programs with data processing systems.
    - ii) Supports computer users throughout the office and manages the local area network.
    - iii) Maintains design and drafting software and provides support to users in the office.

### 3) Bridge Planning and Hydraulics Section

Responsible for program, cost estimates, preliminary bridge plan activities for Trunk Highways and review of state aid bridges. Also, responsible for providing statewide hydraulic engineering services that include design, construction and maintenance activities. In addition, the section provides leadership in the development and implementation of hydraulic automation technology, establishes policy pertaining to hydrology and hydraulics, prepares design aids, provides client training, participates in research projects, and represents the department on state and national committees.

#### a) Agreements and Permits Unit

- i) Selects and negotiates with consulting engineers and administers engineering agreements for the preparation of bridge plans.
- ii) Provides liaison between the Bridge Office and the consulting engineer retained to prepare bridge plans.
- iii) Coordinates public and private utility requirements for bridges.

#### b) Preliminary Plans Unit

- i) Conducts preliminary studies from layouts and develops preliminary bridge plans.
- ii) Provides liaison with District and Central Office road design through the design stage.
- iii) Obtains required permits from other agencies for bridges.

#### c) Hydraulics Unit

- i) Develops and maintains Drainage Manual, standards and specifications related to drainage design and products for use by Mn/DOT and other agencies.
- ii) Provides technical assistance to Districts on all aspects of drainage design.
- iii) Provides bridge and culvert waterway designs for trunk highway projects. Conducts channel surveys for requested waterway bridges.
- iv) Analyzes and evaluates bridges for scour, monitors bridges for scour during floods, and provides training and support for scour monitoring.
- v) Provides technical assistance to counties and municipalities upon request.
- vi) Provides training in hydrology and hydraulics.
- vii) Reviews and prorates cost of storm drains on the municipal and county state aid system.
- viii) Develops, implements, and supports a hydraulic information system to facilitate the sharing of hydraulic data among all users and stakeholders.

- d) Programs and Estimates Unit
  - i) Prepares preliminary, comparative, and final cost estimates.
  - ii) Maintains and provides current program and plan status records.
- 4) Bridge Construction and Maintenance Section

Responsible for bridge construction and maintenance specifications, and bridge construction and maintenance advisory service activities to the office and to the job site.

  - a) Construction and Maintenance Unit; North, Metro and South Regions
    - i) Provides construction and maintenance advisory service to bridge construction and maintenance engineers in the field.
    - ii) Writes bridge construction and maintenance specifications, manuals and bulletins.
    - iii) Writes and maintains the file of standard current special provisions for bridge construction and maintenance.
    - iv) Performs preliminary, periodic and final review of bridge construction and maintenance projects and makes recommendations.
    - v) Reviews bridge plans and special provisions prior to lettings and makes constructability recommendations.
    - vi) Aids municipal and county engineers with bridge construction and maintenance problems, upon request.
    - vii) Provides foundation design including selection of pile type, length, design load, and foundation preparation.
    - viii) Reviews bridge rehabilitation, improvement, and preservation projects and prepares recommendations for scope of work.
    - ix) Aids the Districts in prioritizing upcoming bridge related projects.
    - x) Develops and provides bridge construction trainings for District, county, and municipal bridge construction inspectors.
  - b) Bridge Ratings Unit
    - i) Makes bridge ratings and load postings analysis for new and existing bridges and maintains the records.
    - ii) Reviews and approves special load permit requests.
  - c) Structural Metals Inspection Unit
    - i) Provides inspection services for structural metals, fabrication and assembly to ensure conformity with plans and specifications.

- d) Fabrication Methods Unit
  - i) Reviews and approves structural metals shop drawings submitted by fabricators.
  - ii) Provides fabrication advisory service to designers, fabricators and field construction and maintenance personnel.
  - iii) Provides overhead sign design services to the Office of Traffic Engineering, including the design of bridge-mounted sign trusses.
- e) Bridge Data Management Unit
  - i) Maintains inventory and inspection data for the 19,600 bridges in Minnesota. Works with all agencies to make certain appropriate data is collected.
  - ii) Responsible for implementing bridge management systems to provide information on bridges for maintenance, repair, rehabilitation and replacement.
- f) Bridge Inspection Unit
  - i) Provides expert assistance to the Districts in organizing and conducting inspections of complex bridges, special features, and fracture critical bridges.
  - ii) Conducts quality assurance inspections of all agencies responsible for bridge inspections in Minnesota.
  - iii) Reviews, recommends and provides bridge inspection training for District, county, and municipal bridge inspectors.

For more information, visit the Bridge Office Web site at:  
<http://www.dot.state.mn.us/bridge/>.

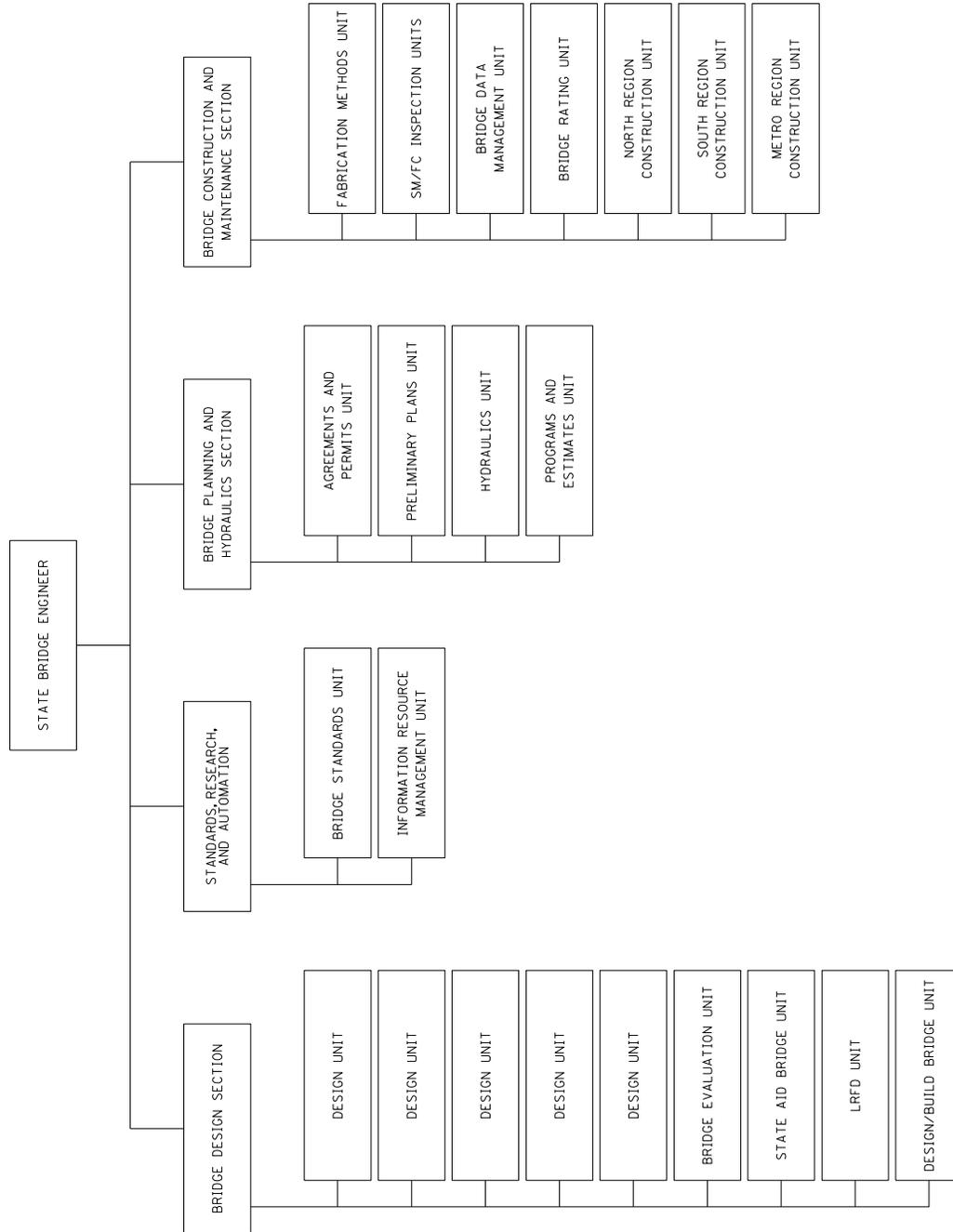


Figure 1.2.1.1 Mn/DOT Bridge Office Organization Chart



Mn/DOT BRIDGE OFFICE ORGANIZATION CHART

BRIDGE OFFICE



### **1.2.2 Highway Systems**

Highways throughout the nation are divided into systems. These system designations are important to know because design standards can vary between the systems. The various highway systems are classified according to the Agency that has responsibility for their improvement, maintenance and traffic regulation enforcement. Listed below are the jurisdictional divisions in Minnesota:

1) Trunk Highway System

The Trunk Highway System consists of all highways, including the Interstate routes, under the jurisdiction of the State of Minnesota. These routes generally are the most important in the state, carry the greatest traffic volumes, and operate at the highest speeds.

2) County Highway System

The County Highway System is made up of those roads established and designated under the authority of the county board. They generally are the more important routes within a county that are not on the Trunk Highway System.

3) Township Road System

The Township Road System is made up of the roads established under the authority of the town board. They generally are of local importance.

4) Municipal Street System

The Municipal Street System is all roads within a municipality not designated as a trunk highway or county road. They are generally of local importance.

### **1.2.3 Bridge Numbers**

All publicly owned bridges, either on or over a trunk highway, that are 10 feet or more in length measured along the centerline of the highway, are assigned a number for identification and cost accounting purposes.

The numbering scheme followed in assigning bridge numbers depends on the time of construction. With few exceptions, the numbering procedure is as follows:

1) Prior to about 1950, all bridges were numbered consecutively from 1 to 9999 as they were constructed. The 8000 series was used for culverts over 10 feet in length (measured along the centerline of the highway). The 7000 series was reserved for county bridges at trunk highway intersections. Five-digit bridge numbers beginning with L or R designate bridges in local bridge systems.

2) Since about 1950, a five-digit number has been assigned to each bridge as it was constructed. The first two digits coincide with the county number (01-87) in which the bridge is located (99 refers to temporary bridges). The last three digits are assigned consecutively using the following guidelines:

- a. 001-499 are used for regular trunk highway bridges.
  - b. 500-699 are used for county bridges.
  - c. 700-999 are used for interstate bridges (any bridge on or over the interstate system).
- 3) In 1991, additional numbers were required for bridges on the state aid system in Hennepin County and for interstate bridges in Hennepin County. To allocate more numbers for bridges on the local system an alpha character is used as the third character of the bridge number. For example, the next bridge number after Bridge No. 27699 will be Bridge No. 27A00. Note that this happens only after 500 and 600 series have been exhausted.

To allocate more numbers on the Interstate road system, the 400 series of numbers will be used along with the 700, 800, 900's presently used. For a bridge number XXYZZ, the following now applies:

- XX = County identification number (99 = Temporary Bridge)
- Y = 0, 1, 2, 3, or R, T, U (for Trunk Highway Bridges)
- Y = 4, 7, 8, 9, or V, or W (for Interstate Bridges)
- Y = X and Y (Trunk Highway or Interstate Culverts)
- Y = 5 or 6 or A through H (for non-trunk highway Bridges)
- Y = J through N, and P, Q (for non-trunk highway Culverts)
- ZZ = Sequence number (00 through 99)

As of September, 2006, the following numbering scheme was added for:

- Bridges or culverts without a highway over or under (e.g. pedestrian trail over stream)
- Existing bridges that have not been assigned a bridge number
- Skyways and other miscellaneous structures such as conveyors, pipelines, or buildings

Use the format **RZZZZ** where:

- R = A literal character
- ZZZZ** = Sequence number (0000 thru 9999)

- 4) In cases of twin bridges, a westbound or southbound lane bridge is generally assigned a lower number than an eastbound or northbound lane bridge.

All bridge numbers are assigned by the Bridge Office. A complete listing of all numbered bridges is available in computer printout form entitled "Minnesota Trunk Highway Bridge Log- Statewide Listing". See Table 1.2.3.1 for a listing of the county identification numbers.

**Table 1.2.3.1 Minnesota County Identification Numbers**

County No.	County Name	District
01	Aitkin	1 & 3
02	Anoka	Metro
03	Becker	4
04	Beltrami	2
05	Benton	3
06	Big Stone	4
07	Blue Earth	7
08	Brown	7
09	Carlton	1
10	Carver	Metro
11	Cass	2 & 3
12	Chippewa	8
13	Chisago	Metro
14	Clay	4
15	Clearwater	2
16	Cook	1
17	Cottonwood	7
18	Crow Wing	3
19	Dakota	Metro
20	Dodge	6
21	Douglas	4
22	Faribault	7
23	Fillmore	6
24	Freeborn	6
25	Goodhue	6
26	Grant	4
27	Hennepin	Metro
28	Houston	6
29	Hubbard	2
30	Isanti	3
31	Itasca	1, 2 & 3
32	Jackson	7
33	Kanabec	3
34	Kandiyohi	8
35	Kittson	2
36	Koochiching	1 & 2
37	Lac Qui Parle	8
38	Lake	1
39	Lake of the Woods	2
40	Le Sueur	7
41	Lincoln	8
42	Lyon	8
43	McLeod	8
44	Mahnomen	2 & 4

County No.	County Name	District
45	Marshall	2
46	Martin	7
47	Meeker	8
48	Mille Lacs	3
49	Morrison	3
50	Mower	6
51	Murray	8
52	Nicollet	7
53	Nobles	7
54	Norman	2
55	Olmsted	6
56	Otter Tail	4
57	Pennington	2
58	Pine	1
59	Pipestone	8
60	Polk	2
61	Pope	4
62	Ramsey	Metro
63	Red Lake	2
64	Redwood	8
65	Renville	8
66	Rice	6
67	Rock	7
68	Roseau	2
69	St. Louis	1
70	Scott	Metro
71	Sherburne	3
72	Sibley	7
73	Stearns	3
74	Steele	6
75	Stevens	4
76	Swift	4
77	Todd	3
78	Traverse	4
79	Wabasha	6
80	Wadena	3
81	Waseca	7
82	Washington	Metro
83	Watonwan	7
84	Wilkin	4
85	Winona	6
86	Wright	3
87	Yellow Medicine	8

#### ***1.2.4 Limit States to Consider in Design***

Bridge designs shall typically consider Strength, Service, Extreme Event, and Fatigue limit states. The limit state checks will vary with the component under consideration. Not all elements will require consideration of all limit states. For example, the fatigue limit state need not be considered for concrete deck slabs in multigirder applications.

### ***1.3 Procedures***

This section covers the Bridge Office procedures for checking of bridge plans, scheduling of projects, and revising or creating standards.

#### ***1.3.1 Checking of Mn/DOT Prepared Bridge Plans***

The general practice of most engineering offices is to require that designs they produce be checked before they are reviewed and certified by the "Engineer in Responsible Charge". Although this practice has always been required for structures designed for Mn/DOT, it is recognized that the quality of the checking process often varies according to time restraints, confidence in the designer, and the instructions given to the checker. Therefore, in order to maintain a consistent design checking process the following guidance is given for routine bridge designs.

For more complex or unusual designs, the checker is advised to discuss additional requirements with the design unit leader. Also, the checking process described is not meant to apply to the check or review functions required for Mn/DOT review of consultant plans (see Section 1.3.2.) or for construction false work reviews. (See the Bridge Construction Manual.)

Three types of design checking will apply:

- 1) An independent analysis of the completed design.
- 2) A check of original design computations for mathematical accuracy, application of code, and accepted engineering practice.
- 3) A review of drafted details for constructibility and accepted engineering practice.

Generally, an independent analysis to confirm the adequacy of the complete design is preferred. Significant differences should be discussed and resolved before the plan is certified. The separate set of calculations should be included with the design file as a record of the completed design check.

When circumstances prevent a complete independent analysis, as a minimum, an independent analysis shall be completed for the following:

- 1) Live and dead loads
- 2) Controlling beam lines
- 3) A pier cap

- 4) A pier footing
- 5) Main reinforcement for high abutments
- 6) An abutment footing

However, for the elements not independently analyzed, the original computations should be checked for mathematical accuracy of original design computations, applications of code, and accepted engineering practice. Checked computations should be initialed by the checker, and the independent analysis should be included in the design file.

When doing a separate analysis, the checker may make simplifying assumptions to streamline the checking process. However, when major differences are found, results must be discussed and resolved with the designer. For instance, for normal piers, piling might be analyzed for dead and live loads only if lateral loads appear to have been reasonably applied in the original computations or the "AISC Beam Diagram and Formula Tables" may be used to approximate pier cap moment and shear.

Whether the check is a completely independent analysis or a minimal analysis combined with a computations check, some details, such as the reinforcing details in a wall corner, also require review by the checker. Often referencing old bridge plans with similar details allows the checker to compare the current design to details that have performed well in the past.

### ***1.3.2 Checking of Consultant Prepared Bridge Plans***

Consultant prepared bridge plans are created by private engineering firms through contracts with the Department or other government agencies. The finished plans are complete to the extent that they can be used for construction.

The Engineer of Record is responsible for the completeness and accuracy of the work. Final design calculations and plan sheets must be completely checked and reconciled prior to submittal. Review comments from the State do not relieve the Engineer of Record of the responsibility for an accurate and complete bridge plan.

Since these plans receive the signature of the State Bridge Engineer, there must be assurance that the plans are geometrically accurate and buildable; structural design is adequate and design codes have been correctly applied; proper direction is given to the construction contractor; and all construction costs are accounted for. Plan errors may cause costly construction delays or safety may be compromised by an inadequate design.

To keep consultant plan reviews consistent and timely, a procedure was developed as a guide that assigns priority to specific items in the plans. The overall review includes "a Thorough Check" and "Cursory Review" of various items. The distinction between "Thorough Check" and "Cursory Review" is as follows:

Thorough Check refers to performing complete mathematical computations in order to identify discrepancies in the plans, or conducting careful comparisons of known data and standards of the Project with values given in the plan.

Cursory Review refers to a comparative analysis for agreement with standard practice and consistency with similar structures, all with application of engineering judgment. Mathematical analysis is not required, but may be deemed necessary to identify the extent of a discrepancy.

The review procedure is listed on the CONSULTANT BRIDGE PLAN REVIEW form following this section. Headings on this list are defined as follows:

**PARTIAL PLAN:** In order to assure that the consultant is proceeding in the right direction, an early submittal of the plan is required. This submittal usually consists of the General Plan and Elevation sheet showing the overall geometry of the structure and the proposed beam type and spacing; the Bridge Layout Sheet; the Framing Plan sheet; and the Bridge Survey sheets. Errors and inconsistencies found in this phase can be corrected before the entire plan is completed. For example, a framing plan, including the proposed beams, must be assured as workable on the partial plan before the consultant gets deep into the design of the remainder of the bridge.

**FINAL PLAN:** A final plan should be complete in all areas to the extent that it can be certified by the designer, although a certification signature is not required for this phase.

**THOROUGH CHECK:** Items indicated for checking on the consultant's partial plan must be correct. Given geometry must fit the roadway layout. Most of this information can be checked using data from the approved preliminary plan. Approval of the partial plan will indicate that Mn/DOT is satisfied with the geometry and proposed structure, and the consultant may proceed with further development of the plan. For the final plan, obvious drafting and numerical errors should be marked to

point out the errors to the consultant; however, the reviewer should not provide corrections to errors in the consultant's numerical computations. Checking on the final plan should be thorough to eliminate possible errors that may occur, such as the pay items in the Schedule of Quantities. Plan notes and pay items can be difficult for a consultant to anticipate because of frequent changes by Mn/DOT. Pay items must be correct because these are carried throughout the entire accounting system for the Project. Plan (P) quantities must also be correctly indicated.

**CURSORY REVIEW:** Normally, a cursory review would not require numerical calculations. This type of review can be conducted by reading and observing the contents of the plan in order to assure the completeness of the work. The reviewer should be observant to recognize what looks right and what doesn't look right. Obvious errors or inconsistencies on any parts of the plan should be marked for correction.

Although structural design is usually the major focus of any plan, most consultants are well versed in design procedures and should need only minimal assistance from the Bridge Office. A comparison of the consultant's calculations with the plan details should be performed to assure that the plans reflect their design and that the applicable codes are followed. An independent design by the Bridge Office is time consuming and is not recommended unless there is a reasonable doubt as to the adequacy of the consultant's design.

**NO REVIEW:** A thorough review of these items would be time-consuming and may not produce corrections that are vital to construction; therefore, it is recommended that little or no time be spent on the listed items. Numerous errors can occur in the Bills of Reinforcement and quantity values. However, checking this information is also time-consuming, hence the burden of providing correct data should be placed on the consultant.

## CONSULTANT BRIDGE PLAN REVIEW

Br. No. \_\_\_\_\_ RTE \_\_\_\_\_ DATE: PARTIAL PLAN REC'D. \_\_\_\_\_ DATE FINAL PLAN REC'D. \_\_\_\_\_  
 DESIGN GROUP \_\_\_\_\_ CONSULTANT \_\_\_\_\_  
 No. OF SHEETS IN PLAN \_\_\_\_\_ DESCRIBE COMPLEXITY \_\_\_\_\_  
 EST. REVIEW TIME BY DESIGN GROUP \_\_\_\_\_(hrs.) ACTUAL REVIEW TIME \_\_\_\_\_(hrs)

PARTIAL PLAN		FINAL PLAN	
THOROUGH CHECK		THOROUGH CHECK	
	Horizontal and vertical clearances		Pay items and plan quantities
	Stations and elevations on survey line		Project numbers
	Deck and seat elevations at working points		Design data block & Rating on GP&E sheet
	Deck cross-section dimensions		Job number
	Working line location and data		Certification block
	Coordinates at working points and key stations		Standard plan notes
	Substructure locations by station		Concrete mix numbers
	Framing Plan		Construction joint locations
	Conformance to preliminary plan		Prestressed beam design if inadequate design is suspected
	Design loads		Bridge seat elevations at working points
			Utilities on bridge
			Existing major utilities near bridge
		<b>CURSORY REVIEW</b>	
			Steel beam splice locations and diaphragm spacing; flange plate thickness increments (enough to save 800+ # of steel)
			Abutment and Pier design to be checked against consultant's calculations
			Conformance to foundation recommendations.
			Pile loads and earth pressures. Check against consultant's calculations.
<b>CURSORY REVIEW</b>			
	Proposed precast beams [per p.5-29]		Rebar series increments (min. 3")
	Precast conformance to industry standards		Interior beam seat elevations
	Proposed steel beam sections		Bottom-of-footing elevations (for adequate cover)
			Railing lengths and metal post spacing (check for fit)
			Use of B-details and standard plan sheets
			Conformance to aesthetic requirements
			Notes – General, construction, reference, etc.
			Quantity items on tabulations
			Precast beam design (Check against consultant's calculations)
		<b>NO CHECK OR REVIEW REQUIRED</b>	
			Diagonals on layout sheet
			Figures in Bills of Reinforcement
			Bar shapes and dimensions
			Rebar placement dimensions
			Bar marks on details against listed bars
			Quantity values (including total of tabulations)

**1.3.3 Peer Review  
for Major or  
Specialty Bridges**

Major bridges are generally defined as bridges containing spans 250 feet and greater in length. A major or specialty bridge may be determined by its type of design, including overall size (length, width, span length, or number and configuration of spans), cost, complexity, feature crossed, security concerns, pier size or shape, or unusual site or foundation conditions. Additionally, the Bridge Design Engineer may elect to require a peer review for unique bridge types. The bridge type will be evaluated by the Preliminary Plans Engineer and the Bridge Design Engineer to determine if it should be considered a major or specialty bridge. Upon concurrence with the State Bridge Engineer, a notation of "Major Bridge" or "Specialty Bridge" will be indicated on the approved preliminary plan. For major bridges designed by consultants, Mn/DOT will require an independent peer review of the design by a second design firm.

Peer review requirements will be described in the Request for Proposal for consultants.

An exception to this requirement is steel plate girder bridges, where review will continue to be performed by in-house design units. See the Bridge Design Engineer for consultation on these requirements.

Once the determination has been made that a particular bridge falls into the category of "Major Bridge" or "Specialty Bridge," an independent design review will be required as part of the original design. This design review may be performed by either in-house Bridge Office staff qualified to review the particular type of design, or by a consultant. Specific design elements for review will be detailed in each contract.

The Engineer of Record is responsible for the completeness and accuracy of the work. Final design calculations and plan sheets must be completely checked and reconciled prior to submittal. Review comments from the State or Peer Reviewer does not relieve the Engineer of Record of the responsibility for an accurate and complete bridge plan. The Engineer of Record will cooperate with the Peer Reviewer as part of the project team.

The Peer Reviewer will participate as part of the project team from the beginning of design to understand the assumptions and develop a relationship with the Engineer of Record.

The following stages of design will be reviewed by the Peer Reviewer for concurrence:

- Design and Load Rating Criteria: Design specifications, construction specifications, design loads and load combinations, construction loads for design, materials and allowable stresses, foundation type, factored pile resistance and resistance factors, and permit trucks.
- Concept Design: Bridge geometrics, typical sections and dimensions, component sizes, framing plan, location and type of expansion joints, location and type of bearings, computer models for girder design, construction staging, construction sequence, river foundation report, vessel impact study, and outline of special provisions.
- Superstructure Final Design: Independent calculations and design; method of analysis (line girder or three dimensional); modeling assumptions; composite and non-composite section properties, member capacities, dead load and live load moments, shears, and stresses at 1/10th points along girder lines, all primary connections and other points of interest; dead and live load deflections; deck design; deck stresses; and deck pour sequence.
- Substructure Final Design: Independent calculations and design, assumptions, points of fixity, cofferdam design, and pier design and details.
- Constructability: Shipping limitations, erection sequence and stability issues, crane sizes and boom lengths, construction overhead clearances, interference/restrictions on construction due to site conditions, shoring tower locations, falsework review.
- Plan: adequacy of construction plans and specifications provided to contractor.
- Load Rating: Independent load rating analysis, rating for moment and shear at 1/10<sup>th</sup> points and any other points of interest of each span.

For each of the stages of design listed above, the Peer Reviewer will submit a Summary of Review Comments, which will be kept by the Peer Reviewer and will verify that the design is feasible and adequately incorporates the Design and Load Rating criteria and Concept Design parameters. The Peer Reviewer may recommend modifications that improve cost-effectiveness or constructability of the design along with Summaries of Review Comments for Design and Load Rating criteria and Concept Design.

The Peer Reviewer will perform reviews at the 30% (Concept Design), 60% (Final Design), and 95% (Plan/Constructability and Load Rating) completion stages using independent design computations as required.

The Peer Reviewer will conduct the final design review without the aid of the original design calculations. The Peer Reviewer will use structural design/analysis software different than that used in the original design—when available—by the Engineer of Record. This will result in a separate set of design calculations—performed by the Peer Reviewer—that will be documented in a report that will be certified. The report will then be compared to the original design performed by the Engineer of Record. The Peer Reviewer will note any changes or recommendations and provide the results to Mn/DOT for review.

The results of the peer review will determine that the design and plans are in compliance with design standards and the established design criteria. The Bridge Design Engineer will resolve issues with the Engineer of Record and Peer Reviewer.

**1.3.4 Schedule for Processing Construction Lettings**

To meet the Department’s schedule requirements for construction lettings, the following schedule for processing bridge plans, special provisions and estimates must be followed. This schedule applies to all projects: Federal Aid, State Funds and Maintenance. In general, processing of bridge plans, special provisions, and estimates for lettings shall be given priority over all other work, and every effort must be made to complete the processing in advance of the times shown, which are deadlines.

**Table 1.3.4.1  
Schedule and Deadlines for Bridge Project Submittal**

Schedule and Remarks	Deadline Time Before Letting Date		
	Federal Project (Full Oversight)	Federal Project	State Project
Final plan and special provisions to 95% completion level.	14 Weeks (Friday)	14 Weeks (Friday)	12 Weeks (Friday)
Preliminary bridge pay items, special provisions, and quantities to Bridge Estimates Unit	13 Weeks (Friday)	13 Weeks (Friday)	11 Weeks (Friday)
Bridge special provisions review complete (by Bridge Construction Unit)	13 Weeks (Friday)	13 Weeks (Friday)	11 Weeks (Friday)
Bridge special provisions completed and sent to Special Provisions & Final Processing Unit (Technical Support)	12 Weeks (Friday)	12 Weeks (Friday)	10 Weeks (Friday)
Bridge plans certified and given to Information Resource Management Unit for dating and distribution to Office of Technical Support	12 Weeks (Friday)	12 Weeks (Friday)	10 Weeks (Friday)
Final bridge pay items and quantities to Bridge Estimates Unit	12 Weeks (Friday)	12 Weeks (Friday)	10 Weeks (Friday)
Final Engineer’s estimate to Cost Estimation Unit (Technical Support)	during 10th week	during 8th week	during 8th week
Latest date for final bridge plan to Special Provisions & Final Processing Unit (Technical Support)	9½ weeks (Tuesday)	8 weeks (Friday)	8 weeks (Friday)

**Table 1.3.4.1  
Schedule and Deadlines for Bridge Project Submittal (Continued)**

Schedule and Remarks	Deadline Time Before Letting Date		
	Federal Project (Full Oversight)	Federal Project	State Project
PS&E package for authorization request to Division Office FHWA	8 <sup>1</sup> / <sub>2</sub> weeks (Tuesday)	7 weeks (Friday)	N/A
Final advertisement	5 Weeks (Friday)	5 Weeks (Friday)	5 Weeks (Friday)
Sale of plans and proposals	4 Weeks (Friday)	4 Weeks (Friday)	4 Weeks (Friday)
Last date for mailing letter addendums by Special Provisions & Final Processing Unit (Technical Support)	10 days (Wednesday)	10 days (Wednesday)	10 days (Wednesday)

**1.3.5 Bridge  
Project Tracking  
System**

Completing a bridge design project for contract letting is a multiple step process that involves input from a variety of work units and personnel. Projects are tracked by Mn/DOT using the Program and Project Management System (PPMS). Within PPMS, projects are divided into activities and the activities are further divided into work tasks. For example, Activity 1260 is "Preliminary Structure Plans" and Work Task 2 of Activity 1260 is "Draft Preliminary Bridge Plan". Progress of the work tasks on active bridge projects is updated monthly.

Following are tables that list work tasks for the major bridge activities within PPMS. Table 1.3.5.1 contains a listing of the PPMS work tasks for Activity 1260, "Preliminary Structure Plans". Tables 1.3.5.2 and 1.3.5.3 contain listings of the PPMS work tasks for Activity 1270, "Final Structure Plans".

For more information on activities and work tasks within PPMS, refer to the PPMS Activity Manual located on the Mn/DOT internal web site at <http://ihub.ots/projdev/pmu/ppms/>.

**Table 1.3.5.1  
PPMS Work Tasks for Mn/DOT or Consultant Prepared Preliminary  
Bridge Plans (Activity 1260)**

Number	Work Task	Percent of Activity Completed
1	Receive and review information (grades, alignment, surveys, layout, Hydraulics report, Project Design Memo., Environmental report)	15%
2	Draft Preliminary Bridge Plan	60%
3	Check Preliminary Bridge Plan	75%
4	Prepare Aesthetics Recommendation	80%
5	Receive and Plot Borings	85%
6	Receive Foundation Recommendations from Regional Bridge Construction Engineer	88%
7	Obtain State Bridge Engineer's Signature	90%
8	Distribute Signed Plans & Distribute Responses on Need for Signs, Lighting, TMC	91%
9	Preliminary Estimate and District Letter	95%
10	Obtain FHWA Approval	99%
11	Turn Over and Meet with Final Design	100%

**Table 1.3.5.2  
PPMS Work Tasks for Mn/DOT Prepared Final  
Bridge Plans (Activity 1270)**

Number	Work Task	Percent of Activity Completed *
1	Receive Preliminary Bridge Plan, Final Repair Recommendation, or Special Structure Request	5%
2	Receive District Design Information (Signal, Lighting, Signing, TMS, etc.)	10%
	Receive Utility Information	
	Receive Stage Construction Sheets	
3	Establish Geometrics	20%
4	Conduct Analysis and Design, Including Check	45%
5	Draft and Check Plan Sheets	75%
	Incorporate Standard Detail Sheets	
6	Construction Unit Review	80%
7	Figure Quantities	85%
8	Send Informational Copies to FHWA and District	88%
9	Final Check of Plan Set by Unit Leader	90%
10	Frame Special Provisions	95%
11	Final Revisions and Check of Plan Set	99%
12	Obtain State Bridge Engineer's Signature	100%

\* May vary by job complexity.

**Table 1.3.5.3  
PPMS Work Tasks for Consultant Prepared Final  
Bridge Plans (Activity 1270)**

Number	Work Task	Percent of Activity Completed
1	Consultant Kick-Off Meeting	25%
2	Partial Plan Delivery and Review	45%
3	Final Plan Delivery and Review	85%
4	Submit for Signature	100%

A listing of the work type codes used in PPMS is given in Table 1.3.5.4.

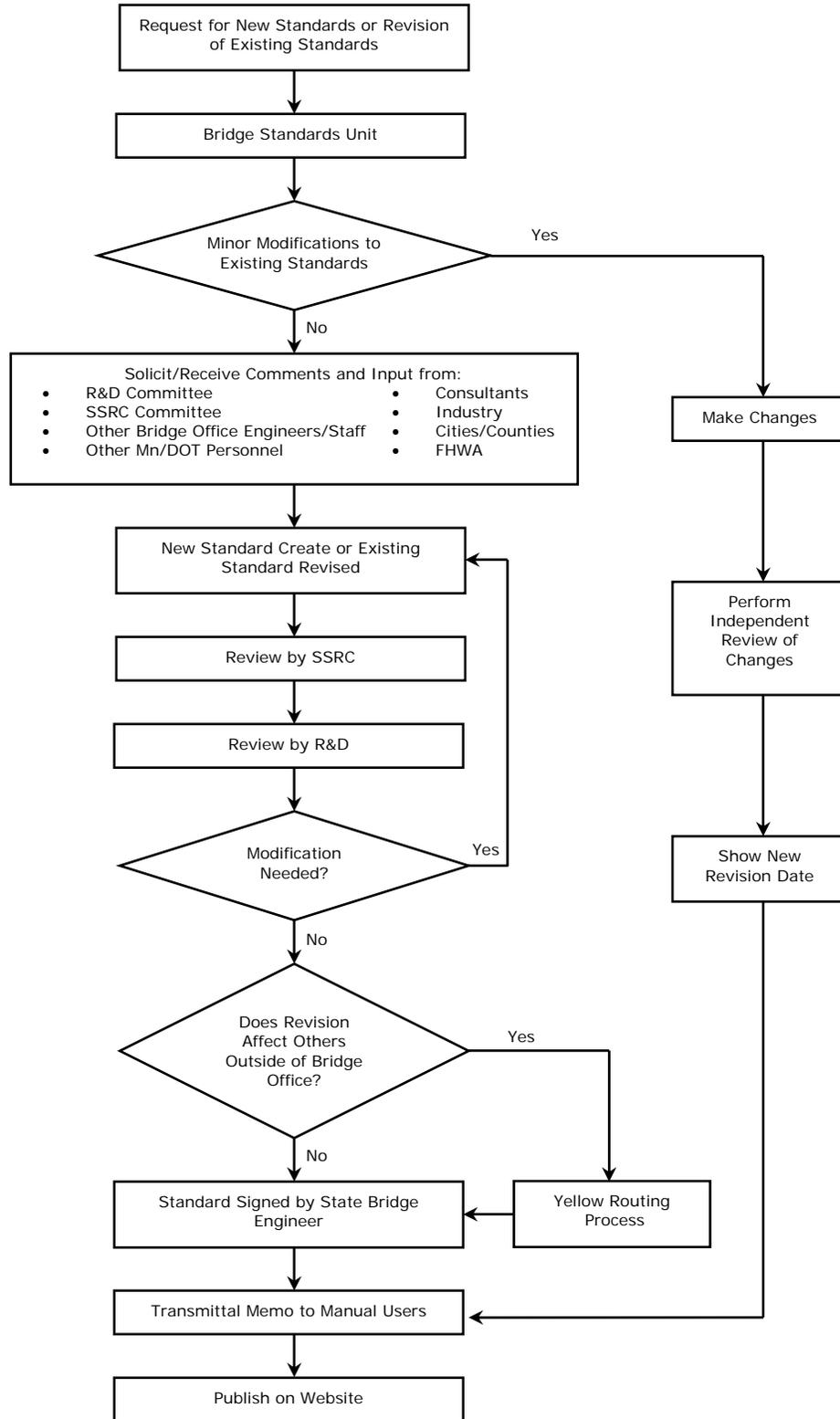
**Table 1.3.5.4**  
**PPMS Bridge Work Type Codes**

<b>Work Type</b>	<b>Description</b>
01	New Bridge
1A	New Bridge (Phase 1) (Early Steel or Stage Construction)
1B	New Bridge (Phase 2)
02	Culvert
2X	Culvert Extension
2B	Concrete Arch
03	Temporary Bridge
04	Pedestrian Bridge
05	Renovation
06	Widen w/Substructure Work
6A	Widen w/Substructure Work (Phase 1) (Early Steel or Stage Constr.)
6B	Widen with Substructure Work (Phase 2)
6T	Temporary Widening
07	Widen without Substructure Work
08	Bridge Length/Short
09	Replace Deck
10	Deck Overlay
11	Replace Railing or Median Barrier
12	Bridge Painting
13	Substructure Repair
14	Remove Bridge
15	Miscellaneous
16	Raise Bridge
17	Replace Superstructure - No Preliminary Plan Required
18	Repair Railing or Median Barrier
19	Replace Joints
20	Deck Repair
21	Rehab or Replace
23	Widen without Substructure Work & Replace Deck
24	Widen without Substructure Work & Deck Overlay
25	Widen without Substructure Work, Deck Overlay & Paint
26	Widen without Substructure Work & Other Minor Work
27	Widen without Substructure Work & Paint
28	Replace Deck & Paint
29	Replace Deck & Other Minor Work
31	Deck Overlay & Replace Railing or Median Barrier
32	Deck Overlay, Replace Railing or Median Barrier & Paint
33	Deck Overlay & Other Minor Work

**Table 1.3.5.4**  
**PPMS Bridge Work Type Codes (Continued)**

<b>Work Type</b>	<b>Description</b>
35	Deck Overlay & Paint
37	Replace Railing or Median Barrier & Paint
38	Replace Railing or Median Barrier & Other Minor Work
39	Paint & Other Minor Work
40	Repair Railing or Median Barrier & Replace Joints
41	Widen without Substructure Work, Replace Deck & Paint
42	Replace Railing or Median Barrier & Replace Joints
44	Deck Repair & Replace Joints
45	Deck Overlay & Repair Railing or Median Barrier
46	Deck Overlay, Repair Railing or Median Barrier & Replace Joints
47	Deck Repair - Rail Rehab
48	Minor Work (Deck Repair, Paint, & Repair Railing or Median Barrier)
49	Deck Overlay, Paint & Repair Railing or Median Barrier
50	Retaining Wall
51	Parking Garage
52	Repair Concrete Arch
54	Riprap
58	Paint & Replace Joints
60	Widen with Substructure Work & Replace Deck
61	Widen with Substructure Work & Deck Overlay
62	Widen with Substructure Work, Deck Overlay & Paint
63	Widen with Substructure Work & Paint
64	Widen with Substructure Work, Replace Deck
66	Widen with Substructure Work & Replace Superstructure
68	Widen with Substructure Work & Replace Railing or Median Barrier
69	Miscellaneous Major
71	Deck Overlay & Replace Joints
91	Probably Bridge
92	Probably Culvert
98	Bridge Scoping
99	Bridge Study

**1.3.6 Approval  
Process for  
Standards**



**Figure 1.3.6.1**  
**Flowchart for Revising Bridge Standards (includes B-Details and Standards)**

## **2. GENERAL DESIGN AND LOCATION FEATURES**

The design of a bridge typically takes place in two major phases of work: preliminary design and final design. During preliminary design, the structure type, the foundation type, the aesthetics, and the primary geometry for the bridge are determined. During final design, specific details for all of the elements of the bridge are developed and presented in the plan set. These details include material descriptions, quantities, and geometric information. Final plan sets are typically assembled in an order that roughly follows the order of construction, from the ground up.

This section of the manual contains a large amount of information useful for the preparation and assembly of plans for a project. To facilitate the production of plans and standardize the content of bridge plan sets, the Bridge Office has developed special provisions, standard bridge details, standard plans, standard plan notes, and standard pay items.

Guidance for the design of specific structural elements (e.g. beams, abutments, piers, etc.) is provided elsewhere in the manual.

### **2.1 Geometrics**

#### **Definitions**

For discussion of bridge geometrics in this section, roadways are classified as Mainline Highways, Ramps, Local Roads, and Local Streets. Each of these four groups is further classified under either Urban or Rural Design.

The following definitions apply:

- Mainline Highways – Roadways that carry through traffic lanes for freeways, expressways, and primary and secondary highways.
- Local Roads – Rural roads off the state trunk highway system.
- Local Streets – Urban roads off the state trunk highway system.
- Ramps – Segments of roadway connecting two or more legs at an interchange.
- Urban Design – Roadways with curbs on the right and/or left sides.
- Rural Design – Roadways without curbs.
- Median Width – The distance between the inside edges of opposing through traffic lanes.
- Auxiliary Lane – A lane adjoining a through traffic lane for a purpose supplementary to through traffic movement such as truck climbing, weaving, speed change or turning.

#### **2.1.1 Bridge Geometrics**

##### **General Criteria**

The width of the bridge deck and the typical section at the bridge undercrossing are determined by the classification and geometrics of the approaching roadway, together with appropriate design considerations for

shoulder needs. The geometrics of the approaching roadway are to be carried over and under the bridge to the maximum extent practicable.

Bridge width requirements are a function of the lane and shoulder widths of the approaching roadway, together with assessment of pedestrian and bicycle needs, multimodal requirements, user safety requirements, drainage requirements, staging, and other project specific considerations such as snow storage and emergency vehicle access. The determination of the appropriate width for each project requires study of specific project needs. Detailed decision documentation is required by the Roadway Designer during the preliminary design phase, and must be coordinated with the Preliminary Bridge Plans Engineer. Bridge shoulder and lane widths should be included with project design element documentation in the District project design memo, including informal design exceptions as necessary.

The discussion of geometric details included in this section describes bridge deck geometrics separately from bridge undercrossing geometrics.

#### **Application of Standards**

Unless stated otherwise, the geometrics discussed in the following articles apply specifically to new work. However, use of these geometrics is also highly desirable when upgrading or widening existing facilities and should be incorporated in those situations also. For bridge repair projects, see the *Bridge Preservation and Improvement Guidelines*, found on the MnDOT Bridge Office web site, for more information. Bridge deck geometrics on the local road system must comply with *State Aid for Local Transportation Operations Rules*, Chapter 8820.

#### **Responsibility**

The Preliminary Bridge Plans Engineer will be responsible for assuring that the geometric standards in this section are followed. Where a deviation from the standard is necessary, a written description of the deviation shall be prepared by the Preliminary Bridge Plans Engineer and submitted to the State Bridge Engineer for approval prior to submitting the Preliminary Bridge Plan for signature.

#### **2.1.2 Bridge Deck Requirements**

##### **Bridge Width Criteria**

Roadway cross sections that approach bridges will normally provide a clear zone recovery area beside the travel lane for the benefit of out-of-control vehicles. It is not economical or practical to carry these full clear zone widths across bridges. Since the railing is most often located within the

clear zone, it is considered a hazard and guardrail protection is required in the approach area.

Roadway shoulder and bridge shoulder width standards have been revised to allow project designers more flexibility, providing them greater latitude to address specific project requirements. For the majority of bridges, the bridge width will match the approach roadway width. For longer and/or more complex bridges, a risk assessment of non-standard width options will be performed to determine the appropriate bridge width. Refer to the document *Performance-Based Practical Design Process and Design Guidance*, found as an attachment to Technical Memorandum No. 18-09-TS-07. Go to: <http://www.dot.state.mn.us/pbpd/design-guidance.html>

Detailed design decision documentation should include a checklist leading to the selected bridge width for the project and must include consideration of the following functions of the shoulder:

- Recovery area to regain control of a vehicle.
- Emergency parking area for stalled vehicles and escape route for stranded motorists.
- Passageway for bicycles and pedestrians.
- Passageway for emergency vehicles.
- Parking area for bridge maintenance and inspection vehicles (working area for under-bridge inspection vehicle and lane closure requirements).
- Temporary traffic lane during deck repairs or overlay construction.
- Area for deck drainage and snow storage.
- Accommodation for passing of wide oversize loads, especially farm machinery.
- Escape area to avoid a head-on collision with an oncoming passing vehicle on a two-lane highway.
- Designated bus shoulder lane.
- Staging needs during construction.

For local roads and streets, bridge widths are given in the *State Aid Manual*, Section 5-892.210 and the *State Aid Operations Rules*, Chapter 8820.

### **Cross Slopes on Bridges**

- 1) Use a cross slope on the bridge traffic lanes that is the same as the approaching roadway lanes, normally 0.02 ft/ft. The shoulder on a bridge may continue at the adjacent lane cross slope or, if better drainage is desired, may be 0.005 ft/ft greater than the adjacent lane. If a shoulder functions as a pedestrian access route, cross slopes must

not exceed 0.02 ft/ft to be ADA-compliant. When the bridge deck is superelevated, provide the same slopes for the shoulders as the adjacent bridge traffic lanes. The 0.005 ft/ft maximum cross slope change between adjacent lanes and shoulders is determined for constructability by limiting the need for atypical detailing such as special bar bends in the deck. Also note that the greater the change in cross slope, the more difficult it is to remove snow to bare pavement. Changes in cross slope between adjacent lanes and shoulders that are greater than 0.005 ft/ft will be considered where steeper slopes will reduce the number of deck drains on the bridge, but must be approved by the Preliminary Bridge Plans Engineer. Note that the effects of a changing cross slope are magnified on curved alignments and require additional consideration and adjustment of stools, seat elevations, and resulting encroachment on vertical clearances.

Keep superelevation transitions off bridges. In instances where they are unavoidable, it is preferable for ease of deck placement to maintain a straight line across the deck at all locations, because it allows a straight screed between paving rails placed at both sides of the deck. Locate begin and end points of transition breaks at piers.

- 2) Provide ramp cross slopes that are uniform between the bridge curbs.

### **Bridge Median**

On divided highways with a separate bridge for each roadway, the openings between bridges must be a minimum of 8'-0" wide if access for bridge inspection vehicles is required.

Use longitudinal joints along the median of bridges only on bridge roadways wider than about 100 feet or for other special cases. By eliminating this joint on bridges with medians, simpler detailing and simpler construction can be used.

### **Shared-Use Paths and Pedestrian Walkways (Sidewalks) on Bridges**

Shared-use paths are provided on bridges where both pedestrian and bicycle traffic is expected. Bridge walkways are provided where only pedestrian traffic is expected.

The width of bridge shared-use paths and walkways are highly dependent on their context (i.e., factors such as land use, user type, expected volume, state and local non-motorized plans, network connections, trip attractions, overlooks, future growth, and bridge length).

When including pedestrian and/or bicycle traffic on a bridge, note that safety, accommodation, and cost must be balanced for all users throughout the roadway cross section. This includes balancing the widths of lanes, shoulders, shared-use paths, and walkways, particularly in constrained cross-sections.

The AASHTO *Guide for the Development of Bicycle Facilities* (GDBF) recommends a minimum two-way shared-use path paved width of 10 feet.

The *Proposed Guidelines for Pedestrian Facilities in the Public Right of Way* (PROWAG) requires a continuous minimum clear public access route (PAR) width of 4 feet and a minimum clear PAR width of 5 feet at intervals of 200 feet to allow for passing.

On bridges, MnDOT also includes a buffer width added on each side of the shared-use path/walkway in order to protect users from vertical barriers and edge of walkway drop-offs.

Use the following guidance for determination of bridge shared-use path/walkway widths. For local bridges, also refer to *State Aid Operation Rules*, Chapter 8820.

1) New vehicular bridges

Best practice is to provide continuity by matching the measured width of the approach shared-use path/walkway, and adding a 1 foot buffer width on each side. See Figure 2.1.2.1. For approach shared-use paths/walkways that are located immediately behind a curb, the approach width is measured to the back side of the curb. Integral brush curbs (maximum of 2 inches wide x 6 inches high) may be included in the clear width dimension where the total width is greater than 10 feet. For approach shared-use paths not meeting the AASHTO GDBF recommended minimum width of 10 feet, consult state or local plans and/or the appropriate trail authority to identify the future intent and feasibility of providing a greater approach path width. Consult with functional group experts as necessary.

Total bridge shared-use path/walkway widths greater than best practice or greater than 12 feet require consultation with the state or local authority and/or the appropriate trail authority to identify the need for additional width. The District and/or local authority must document the need for and feasibility of providing this width (plan, cross section, letter, user volume, etc.). Total widths beyond 12 feet require concurrence from functional group experts and discussion to determine whether municipal cost participation is necessary.

The minimum total bridge shared-use path width for new vehicular bridges is 10 feet, which is based on an 8 foot approach shared-use path (two times the 4 foot PAR width) plus a 1 foot buffer width on each side. Consideration may be given to a minimum total bridge shared-use path width less than 10 feet when the approach shared-use path width is less than 8 feet and/or there is concurrence from functional group experts.

For new vehicular bridges that accommodate pedestrian traffic only, the minimum total bridge walkway width is 7 feet, which is based on the 5 foot PAR width for passing plus a buffer width of 1 foot on each side.

The total bridge shared-use path/walkway width is defined as the minimum clear width measured from the path/walkway side of the curb/barrier/parapet/railing to the path/walkway side of the opposite curb/barrier/parapet/railing. For situations where there is no barrier/parapet on the traffic side of the shared-use path/walkway (raised sidewalk), the measurement is to the top outside edge of the shared-use path/walkway. There, the location of the top outside edge of the shared-use path/walkway is defined as 1 inch from the gutter line (based on 6 inch curb height x 0.125 slope = 0.75 inches, rounded up to 1 inch). Integral brush curbs (maximum of 2 inches wide x 6 inches high) may be included in the clear width dimension where the total shared-use path/walkway width exceeds 10 feet. See Figure 2.1.2.1.

2) New pedestrian bridges

For new pedestrian bridges that carry both pedestrians and bicycle traffic, follow the guidance given in 1) above.

For new pedestrian bridges carrying pedestrians only (note that this is a rare occurrence), the minimum total bridge walkway width is 8 feet per the requirements of AASHTO's *A Policy on Geometric Design of Highways and Streets*.

3) Bridge repair projects

Where possible, follow the guidance given in 1) above for bridge repair projects.

On bridge repair projects with constrained cross-sections, the minimum total bridge shared-use path width is 8 feet.

On bridge repair projects with constrained cross-sections that accommodate pedestrian traffic only, the minimum total bridge walkway width is 5 feet. Consideration may be given to a minimum total width of no less than 4 feet where constrained bridge cross-sections are less than 200 feet long and there is concurrence from functional group experts. See Figure 2.1.2.1.

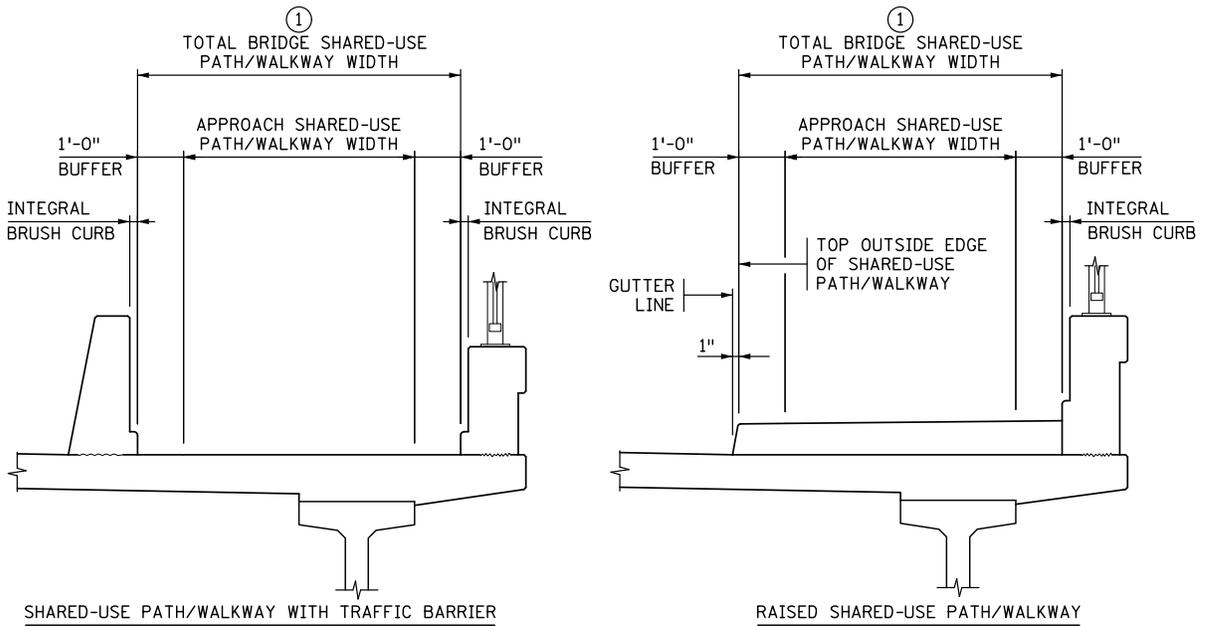
When the design speed on the bridge is 50 mph or greater, a concrete barrier that meets TL-4 is required between the roadway and the shared-use path/walkway. In addition, a pedestrian or bikeway railing is required on the outside of the shared-use path/walkway. For design speeds of 40 mph or less, separation with a concrete barrier is not required. For a design speed of 45 mph, consider the context when determining whether separation is needed:

- Built up urban areas versus open suburban/rural areas.
- Proximity of approach shared-use path/walkway to roadway and whether there is adequate cross-section width to provide and terminate approach guardrail.
- Proximity of intersections to the bridge and whether intersection sight distance will be affected by inclusion of barrier and guardrail.
- Volume of pedestrian/bicycle usage on the bridge.
- Actual operating speed compared to the design speed.
- Horizontal alignment and location of the shared-use path/walkway.

The curb height for shared-use paths/walkways adjacent to the roadway is 6".

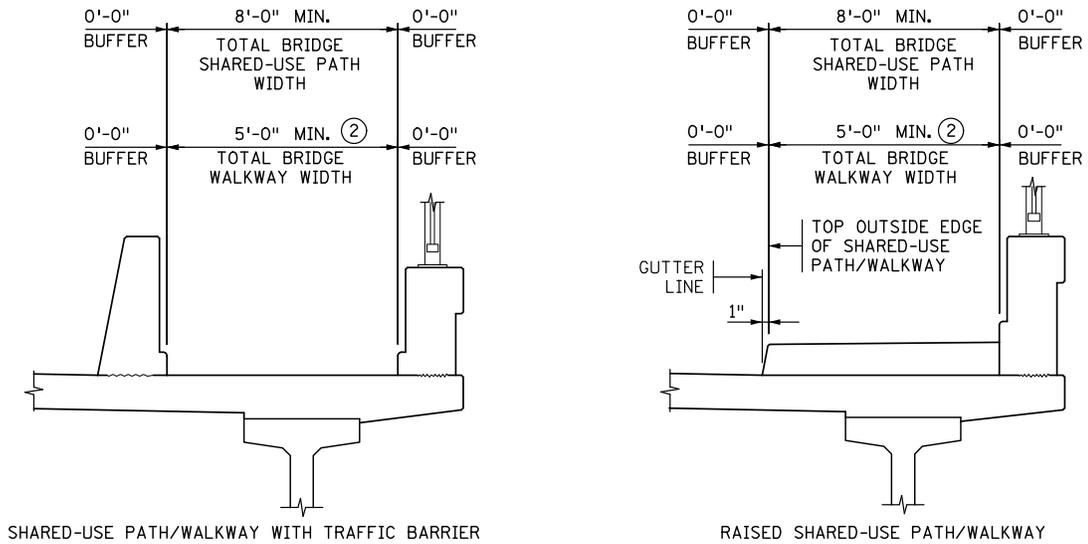
When a barrier is provided between the traffic lanes and the shared-use path/walkway, use the bridge slab for the shared-use path/walkway (i.e., do not provide a raised shared-use path/walkway). Advise the road plans designer to provide for any necessary shared-use path/walkway ramping off the bridge.

The minimum cross slope for shared-use paths/walkways is 0.01 ft./ft.



① FOR TOTAL WIDTHS > 10'-0", INTEGRAL BRUSH CURB (MAX. 2" WIDE x 6" HIGH) MAY BE INCLUDED IN TOTAL WIDTH.

SHARED-USE PATH/PEDESTRIAN WALKWAY WIDTH FOR BRIDGES



② CONSIDERATION MAY BE GIVEN TO A MINIMUM TOTAL WIDTH OF 4'-0" ON CONSTRAINED BRIDGE CROSS-SECTIONS < 200 FT. LONG WITH CONCURRENCE FROM FUNCTIONAL EXPERTS.

SHARED-USE PATHS/PEDESTRIAN WALKWAYS-  
MINIMUM WIDTH FOR BRIDGE REPAIR PROJECTS

FOR CONSTRAINED CROSS-SECTIONS

Figure 2.1.2.1

**Protective Barriers at Bridge Approaches**

The ends of bridge barriers must be protected from being impacted (except on low speed roads such as city streets). For design speeds over 40 mph, a crash tested guardrail transition is required.

Refer to *State Aid Operation Rules*, Chapter 8820 for guardrail requirements on local bridges.

**2.1.3 Bridge Undercrossing Geometrics****General Criteria for Lateral Clearance**

Bridge undercrossing geometrics must rationalize safety requirements with costs and physical controls such as span length and permissible depth of structure. The following guidelines apply in establishing these geometrics:

**1) Safety**

Piers, abutments, side slopes and back slopes steeper than 1:3, and guardrails can all be hazards to an out of control vehicle. It is desirable at all bridge undercrossings to provide a clear zone recovery area beside the roadway that is free from these hazards. This clear zone is given in the *Road Design Manual*, Section 4-6.04 and is a function of the roadway curvature, design speed, ADT, and ground slope. For the area under bridges a practical maximum clear zone of 30 feet may be used as permitted in the *2011 AASHTO Roadside Design Guide*, Table 3.1 based on consistent use and satisfactory performance. Eliminate side piers from the roadside area wherever possible. The "desirable" bridge undercrossing will satisfy the above safety criteria.

For locations where it is totally impractical to provide a full clear zone recovery area at an undercrossing (as at some railroad underpasses and in certain urban situations), lesser side clearances are permitted. Where the full recovery areas must be infringed upon, use the greatest side clearances that circumstances will permit. For example: A side clearance of 20 feet is not as desirable as 30 feet but is still better than the absolute minimum clearance.

Where drainage must be carried adjacent to the roadway passing under a bridge, either a culvert must be provided at the approach fill or the ditch section must be carried through at the toe of the bridge approach fill. The use of a culvert will often permit more desirable bridge geometrics, but the culvert openings can also introduce a roadside hazard, requiring guardrail. A determination regarding drainage (need for culverts, size of a culvert, and assessment of possible hazard) will be a controlling factor in deciding geometrics of the bridge for the site.

## 2) Economics

Prestressed concrete beam spans are normally the most economical type of construction for grade separations. In addition, there will usually be greater economy in constructing grade separations using two long spans rather than constructing four shorter spans.

The span lengths and overall bridge length affect the abutment heights, which in turn affect the overall cost of the bridge. See BDM Article 2.3.2 for discussion of bridge types and their economical/typical span ranges as well as a discussion on abutment and pier type selection.

## 3) Appearance

The use of longer spans will necessitate a deeper superstructure and higher approach fills. Consideration should be given to the effect of the depth of structure on the overall appearance and design of the undercrossing.

For rough calculations during preliminary planning, the depth of highway bridge superstructures can be assumed to be about 1/20 of the length of the longest span. (Depth of superstructure refers to the dimension from top of slab to bottom of beam.) Contact the Preliminary Bridge Plans Engineer for the exact dimensions to be used in final planning and for depth of structure on railroad bridges.

### **Lateral Clearance for Roadways**

The desirable lateral clearance right and left from the edge of through traffic lanes to any hazard (as described above) is the full clear zone. Although guardrail may still be required outside of the bridge limits when the full clear zone cannot be met, 30'-0" may be used as a practical maximum lateral clearance in the area under bridges. Eliminate side piers wherever possible.

### **Lateral Clearance for Railroads**

Lateral clearances at railroads are to be determined as follows:

- 1) The statutory clearances diagram shown on Figure 2.1.3.1 represents the absolute minimums that must be adhered to. For design, use a minimum horizontal clearance of 9'-0" to a pier or abutment (8'-6" is the legal minimum).
- 2) MnDOT and FHWA have agreed to the horizontal clearances shown in Figure 2.1.3.1 (25'-0" minimum clearance to pier, 30'-6" to "back slope control point") for mainline BNSF RR tracks at sites meeting the following conditions:

- a) When the standard will not increase the cost of the structure by more than \$50,000.
  - b) When sufficient vertical clearance exists between the tracks and in-place or proposed roadway profile to accommodate the structure depth necessary for the longer spans typically required by the standard.
- 3) Back slopes shall be 1V:2H and pass through the "back slope control point" shown on Figure 2.1.3.1 for the applicable case. The dimension to the "back slope control point" indicates the maximum extent of federal participation in the construction and must not be exceeded.
- 4) The Preliminary Bridge Plans Engineer will contact the MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO), to negotiate with the railroad the need for provisions for a maintenance road for track maintenance equipment and future track requirements.

#### **Waterway Sections Under Bridges**

The Waterway Analysis (hydraulics report) gives information on the required stream cross section under the bridge including waterway area and low member elevation. Potential flood damage, both upstream and downstream, and permitting agencies' requirements must be considered.

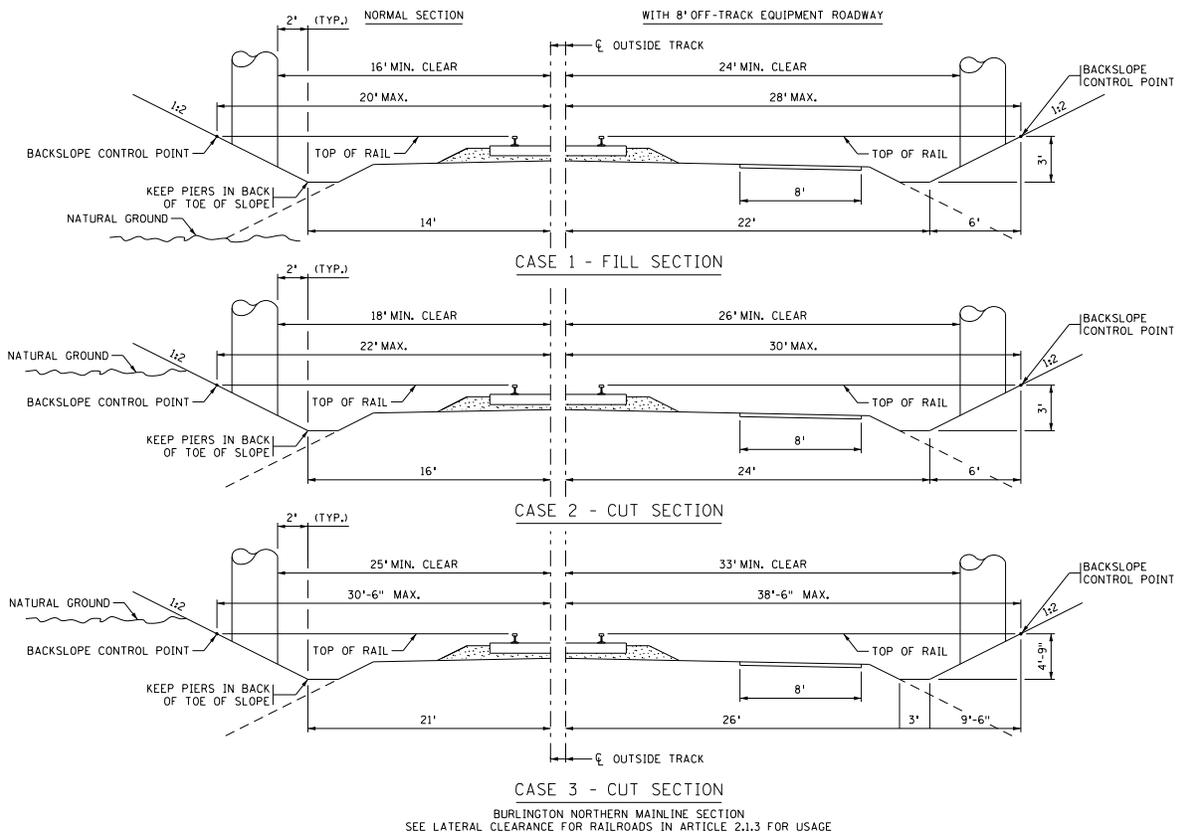
The Preliminary Bridge Plans Engineer will coordinate with the Waterway Unit to determine when a wildlife passage bench is required under bridges over waterways.

For bridges on the local system, go to the State Aid Bridge Web Site at <http://www.dot.state.mn.us/stateaid/bridge/resources.html> and refer to the guidance found under Hydraulics.

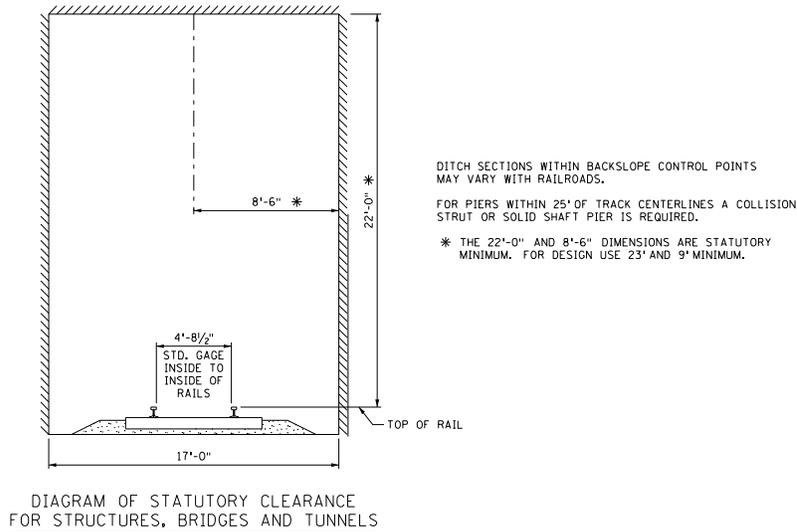
#### **Vertical Clearance for Underpasses**

Vertical clearance requirements are aligned with the 2011 edition of the AASHTO book, *A Policy on Geometric Design of Highways and Streets*, (see page 10-21), which recommends a minimum vertical clearance of 1'-0" above the legal vehicle height, plus an allowance for future pavement resurfacing and other considerations. The legal height of a truck in Minnesota is 13'-6".

Table 2.1.3.1 lists the minimum vertical clearance requirements for trunk highway underpasses.



LOCATION OF PIERS AND BACKSLOPES



RAILROAD CLEARANCES

**Figure 2.1.3.1**  
**General Railroad Clearances**  
*(Note that railroad approval is required for specific project clearances)*

**Table 2.1.3.1 Vertical Clearances for Underpass Bridges**

TYPE OF STRUCTURE	MINIMUM VERTICAL CLEARANCE FOR NEW BRIDGES ①②	MINIMUM VERTICAL CLEARANCE UNDER EXISTING BRIDGES (FOR PAVEMENT RECONSTRUCTION PROJECTS) ③
Trunk Highway Under Roadway or Railroad Bridge (Super Load OSOW Corridors) ④	16'-6"	16'-6"
Trunk Highway Under Roadway or Railroad Bridge	16'-4"	16'-0"
Trunk Highway Under Pedestrian Bridge ⑤	17'-4"	17'-0"
Trunk Highway Under Sign Bridge ⑤	17'-4"	17'-0"
Railroad Under Trunk Highway Bridge ⑥	23'-0"	NA
Portal Clearances on Truss or Arch ⑦	20'-4"	20'-0"

- ① Additional clearance to provide for future resurfacing is desirable and should be provided where practical. Traditional bituminous overlay allowances range from 3" to 6". Un-bonded concrete overlay projects are now well above the 6" tolerance, and can be as high as 12". The appropriate design value will depend on the pavement types, its initial structure type, and lifecycle strategy, and should be coordinated with the Pavement Design Engineer.
- ② A clearance height that includes a future resurfacing allowance may be used in place of the listed minimums, provided the resulting clearance is at least as much as the listed minimums in this column. Construction tolerance requirements have been reviewed and deemed adequate for new bridge construction so long as the value shown for "Minimum Vertical Clearance for New Bridges" is used, rather than AASHTO minimums. Adjust table values upward as required for overlay requirements exceeding 4".
- ③ The minimum vertical clearances shown are the absolute minimum clearances to be achieved after pavement reconstruction, under an existing bridge or structure. These minimums are not to be used as design minimums for new bridges or bridge reconstruction projects. The minimums are only acceptable due to the known spatial location of the existing structure, thus eliminating the construction tolerance risk of that existing element.
- ④ A minimum vertical clearance of 16'-6" is required on designated Super Load OSOW Corridors. Super Load OSOW Corridors are designed to accommodate an envelope size of 16' wide x 16' high x 130' long,

traveling along the corridor. Contact the MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO) for specific corridor locations and requirements (<http://www.dot.state.mn.us/cvo/index.html>).

- ⑤ The additional 12" of vertical clearance under pedestrian and sign bridges is provided because these bridges are much less substantial and could collapse in the event of a hit.
  
- ⑥ Vertical clearance over railroad infrastructure requires approval of the railroad. The 23'-0" clearance above the top of the rails is the minimum clearance required by the American Railway Engineering and Maintenance of Way Association (AREMA) manual. The maximum vertical clearance for Federal Cost Participation is 7.1 meters (23'-4") per the Code of Federal Regulations (see CFR 646 Appendix to Subpart B of Part 646). This is allowed where it is the railroad's standard practice to accommodate future ballasting of the tracks. The State of Minnesota statutory minimum vertical clearance is 22'-0". For clearances below 22'-0", approval from the MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO) and the railroad is required. Contact the Rail Safety and Coordination Project Manager from the OFCVO for assistance with railroad coordination, agreements, and approvals. See Figure 2.1.3.1.
  
- ⑦ Minimum portal clearance values were set based on historical portal heights.

The clearance over highways applies to the traffic lanes and full usable width of shoulders.

Per Minnesota Rules, Chapter 8820, Local State-Aid Route Standards, the minimum vertical clearance for highway underpasses (including construction tolerance) is 16'-4" for rural-suburban designs and 14'-6" for urban designs. For trunk highways crossing local roads or streets at a freeway interchange, the minimum vertical clearance with construction tolerance is 16'-4". A complete list of vertical clearances for local roads and streets is found in the *State-Aid Operations Rules*, Chapter 8820.

Where bikeways pass under a bridge or through a tunnel, a vertical clearance of 10'-0" is desirable for adequate vertical shy distance. (See *AASHTO Guide for the Development of Bicycle Facilities*, 4<sup>th</sup> Edition, pages 5-6 and 5-26.) Where this is impractical to obtain, a lesser clearance down to a minimum of 8'-0" is acceptable. Clearances below 10 feet on the local road system will require a variance to the *State-Aid Operations Rules*, Chapter 8.

### **Vertical Clearance over Waterways**

The location and project description for all bridges over Minnesota waterways are to be reviewed by the U.S. Coast Guard (USCG) for potential permitting requirements.

## 1) Non-Navigable Waterways

A 3'-0" minimum clearance between the 50-year flood stage and low point on the bridge superstructure is recommended. This amount of clearance is desired to provide for larger floods and also for the passage of ice and/or debris. If this amount of clearance is not attainable due to constraints relating to structure depth, roadway grades or other factors, reduced clearance may be allowed. The Preliminary Bridge Plans Engineer, after consultation with the Waterway Unit and the MnDOT District Office, will determine the required clearance.

## 2) Navigable Waterways

a) Examples of waterways that require a construction permit (generally considered to be waterways for commercial shipping) from the USCG include:

- The Mississippi River downstream from the railroad bridge that crosses the river south of 42<sup>nd</sup> Avenue North in Minneapolis (River Mile Point 857.6)
- The Minnesota River downstream from location just west of T.H. 101 river crossing in Shakopee (River Mile Point 25.6)
- The St. Croix River downstream from Taylors Falls
- The St. Louis River downstream from Oliver, Wisconsin.

Guide vertical clearances published by the USCG are:

- Mississippi River:
  - 52.0' above 2% flowline elevation or 60.0' above normal pool elevation, whichever is greater, for the portion downstream of the Burlington Northern Railroad Bridge near the University of Minnesota (River Mile Point 853.0).
  - 21.4' above river stage of 40,000 c.f.s. for the river portion upstream (River Mile Point 853.0 to 857.6).
- Minnesota River:
  - 55.0' above normal pool elevation from the river mouth to I-35W bridge (River Mile Point 10.8).
  - 30.8' above 1881 high water elevation from I-35W bridge (River Mile Point 10.8) to Shakopee (River Mile Point 25.6).
- St. Croix River:
  - 52.0' above 2% flowline elevation or 60.0' above normal pool elevation, whichever is greater, from the river mouth to Stillwater.
- Lake Superior Watershed:
  - Navigation clearances are determined by USCG on a case-by-case basis.

Consult the Preliminary Bridge Plans Engineer when establishing navigation clearances.

b) All Other Navigable Waterways

Bridges that cross waterways in other portions of the state may be required to provide for local pleasure boat traffic. Vertical clearance for these bridges will be determined on an individual basis, based on local needs. The Preliminary Bridge Plans Engineer, after consultation the Waterway Unit, the MnDOT District Office, and the MnDNR, will make this determination based on specific conditions of the waterway.

### **Vertical and Horizontal Alignment**

Information governing vertical curves, horizontal curves, and sight distance may be found in the *Road Design Manual* and *Technical Manual*.

When preparing preliminary bridge plans for the local road system, vertical and horizontal alignment charts from the *State Aid Manual* shall be employed.

#### **2.1.4 Bridge Barriers and Railings**

See Section 13 of this manual for the policy on design of bridge barriers and railings for MnDOT projects.

#### **2.2 Bridge Aesthetics**

The aesthetic design process is initiated early in the bridge planning phase.

The Preliminary Bridge Plans Engineer, the Preliminary Bridge Architectural Specialist, the District, and the financial stakeholders determine the aesthetic design level with an eye on constructability and cost. Other people, offices, agencies, etc. may also be involved. The extent of this involvement may vary depending on the individual project. This process leads to the development of an Aesthetic Plan for the bridge. Once the project reaches the final stage, the Bridge Design Unit Leader implements the Aesthetic Plan to completion with assistance from the Preliminary Bridge Architectural Specialist as needed.

Note that constructability of aesthetic components and complexity of the aesthetic details may affect the project schedule, and therefore must be considered during the development process.

Section 3 of the *Aesthetic Guidelines for Bridge Design Manual* describes the process of aesthetic design in more detail.

Maximum levels of MnDOT participation in aesthetic costs are given in the *Cost Participation and Maintenance Responsibilities with Local Units of Government Manual*.

### **2.3 Preliminary Bridge Plans**

#### **2.3.1 General**

#### **Purpose**

The Preliminary Bridge Plan serves to document the main features of the bridge (type, size, location, aesthetics, etc.) and is used to obtain approvals and coordination before final design begins. By doing this, the time and expense of revising a completed plan will hopefully be avoided. The plan coordinates the work between Road Design and the Bridge Office and enables the cost and scope of the work to be estimated.

Specific users of the plan include:

- Road Designers to verify the grade, alignment and roadway widths and to obtain the approximate limits of grading, paving and guardrail at the bridge ends.
- FHWA to review and approve unusual or complex bridge projects.
- Bridge Office Consultant Agreements Unit to select and negotiate contracts with consultants.
- Final Bridge Design Units and Consultants to prepare final plans.
- Bridge Scoping Engineer and Bridge Estimates Unit to prepare a preliminary estimate of the bridge costs.
- MnDNR, U.S. Coast Guard, U.S. Corps of Engineers, and Watershed Districts to review and issue required permits for stream crossings.
- Cities, Planning Agencies, and citizen groups to review and approve projects.
- District Traffic Engineer and Regional Transportation Management Center (RTMC) to convey their needs on the new bridge.
- MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO) for use in negotiating railroad agreements.

In preparing preliminary bridge plans, the plan users should always be kept in mind, particularly those without bridge technical experience.

#### **Requirements for Preliminary Bridge Plans**

Preliminary bridge plans are required for all new trunk highway bridges (including MnDOT precast concrete arch and three-sided structures and pedestrian underpass box culverts) and all bridge widening projects where substructure widening is required. In addition, preliminary plans signed by the State-Aid Bridge Engineer are required for all county and local bridges that cross a trunk highway. Preliminary bridge plans are not required for culverts (except those used for pedestrian access), overlays,

deck replacements, and other projects where substructures are not widened.

The Bridge Preliminary Plans Unit normally prepares preliminary plans for new trunk highway bridges, although consultants may also develop plans. Preliminary plans for bridge widening projects are normally prepared by the Bridge Design Units since significant design work is required to evaluate the existing structure and schemes for widening and handling traffic.

Preliminary plans prepared by Consultants or Design Units are submitted to the Bridge Preliminary Unit for review, acceptance, submittal to the State Bridge Engineer for signature, and distribution of signed copies.

### **Contents**

The Preliminary Bridge Plan consists of a general plan and elevation sheet, survey sheet, and borings sheet. For complex urban structures additional road design sheets giving alignment, superelevation diagrams, utilities, contours, traffic staging, intersection layout, and aesthetics may be included. The Preliminary Bridge Plan contains: plan and elevation views, a transverse section, design data, data on the type of structure, foundation requirements, and aesthetic treatment. When aesthetics are of special importance, architectural type drawings showing the proposed treatment or type of construction may also be included. For bridge widening projects, the survey sheet may be eliminated or a copy of the survey sheet from the existing bridge may be included.

### **Preparation of Preliminary Bridge Plans**

The steps involved in preparing a typical preliminary plan set for a new trunk highway bridge by the Preliminary Unit are as follows:

- 1) *Request for Bridge Scoping and Cost Estimating Assessment – Bridge Replacement* (Form A) is completed, which provides the initial information for the project. Form A is found at:

<http://www.dot.state.mn.us/bridge/scoping.html>

Consideration is given to the use of Accelerated Bridge Construction (ABC) methods at this step in the process. Results of the ABC Stage 1 assessment is reported on Form A and, if applicable, the ABC Stage 2 assessment is completed and included as an attachment. In addition, a bridge number is requested by the MnDOT District Project Manager or their designee by completing the online *New bridge number request form*, found at:

<http://www.dot.state.mn.us/bridge/bridgereports/index.html>

A new bridge number is then assigned.

- 2) Approved geometric layouts are received from the District.

- 3) Bridge survey sheets are received from the District Surveys Section. Copies are sent to the Foundations Unit of the Office of Materials and Road Research requesting soil borings. For stream crossings, a copy is sent to the Bridge Office Waterway Unit requesting a waterway analysis.
- 4) A depth of structure and span arrangement are determined using the layout and waterway analysis and are given to Road Design. This typically involves communication between the Bridge Office, Road Design, and Hydraulics to arrive at a structure depth and span arrangement that produces the best overall solution. If a railroad is involved, negotiations are held with the railroad to determine what features should be incorporated into the plan to satisfy the railroad's needs and also meet MnDOT standards.
- 5) Final grades and alignment are developed and officially received from Road Design.
- 6) A CADD technician is assigned the project and drafting of the plan begins. Clearances are checked and more exact span lengths determined.
- 7) Traffic data is requested and received from the District Traffic Engineer.
- 8) The extent of aesthetic treatment is determined following the process described in BDM Article 2.2.
- 9) Borings are received electronically from the Foundations Unit and plotted on the survey sheets.
- 10) The Preliminary Bridge Plans Unit checks the completed preliminary package, except the foundation type.
- 11) The preliminary package is given to the Regional Bridge Construction Engineer along with the foundation report for determining pile type, lengths, and resistance. When received, the pile information is added to the preliminary plan.
- 12) The completed Preliminary Bridge Plan is reviewed with the Bridge Planning and Hydraulics Engineer and taken to the State Bridge Engineer for signature.

#### **Time Schedule for Preliminary Plan Preparation**

The time schedule for receiving information and completing preliminary bridge plans for normal bridges, as given in Primavera P6, is shown in Table 2.3.1.1.

**Table 2.3.1.1 Preliminary Plan Time Schedule**

WORK ITEM	TIME PRIOR TO SCHEDULED LETTING DATE
Bridge Survey	21 months
Hydraulics	18 ½ months
Grades and Alignment	18 ½ months
Foundations	17 months
Preliminary Plan Completed	16 months (typical bridges) 20 months (major bridges)

Additional lead-time beyond that given in the table above is required for major bridges, bridges involving agreements with cities or railroads, and bridges with extensive aesthetic requirements.

In addition to the work items listed above, time must be allotted for a formal type selection study for major bridges.

**Use of Preliminary Bridge Plans**

The completed and signed Preliminary Bridge Plan becomes the department’s official proposal for that structure. The following steps are then taken:

- 1) The Bridge Estimating Unit in the Bridge Office prepares an estimated contract construction cost for the structure.
  
- 2) Copies of the Preliminary Bridge Plan are distributed to the various offices of MnDOT and outside agencies for information, review, and approval, as the case may be. (See Table 2.3.1.2.)

Approval by all concerned of the proposed structure dimensions, type of construction, and geometrics before the start of final design is one of the most important functions of the Preliminary Bridge Plan. This is particularly true of stream crossings, railroad crossings (over and under), and structures requiring special aesthetic treatment.

The Federal-Aid Highway Program (FAHP) provides federal-aid to State-selected projects. The Federal Highway Administration (FHWA) administers the FAHP on behalf of the U.S. Secretary of Transportation under Title 23 and therefore is one of the outside agencies that reviews bridge projects. The *FHWA Minnesota Division and Minnesota Department of Transportation Stewardship & Oversight Agreement*

documents the roles and responsibilities of the FHWA and MnDOT regarding project approvals and review:

- For most bridge projects, MnDOT assumes the FHWA's Title 23 responsibilities and only a courtesy copy of the Preliminary Bridge Plan transmittal letter is sent to FHWA (without the plans) for informational purposes.
- For unusual or complex bridges and structures, the FHWA Minnesota Division is responsible for the approval of the Preliminary Bridge Plan. For the purpose of this guidance, unusual or complex bridges and structures are defined as those that the FHWA Minnesota Division determines to have unique foundation problems, new or complex designs, exceptionally long spans, exceptionally large foundations, complex hydrologic aspects, complex hydraulic elements or scour related elements, or that are designed with procedures that depart from currently recognized acceptable practices. Examples of unusual or complex bridges and structures include cable-stayed bridges, suspension bridges, arch bridges, segmental concrete bridges, movable bridges, truss bridges, tunnels, complex geotechnical wall systems, and complex ground improvement systems.

When submitting preliminary documents to the FHWA, include the Preliminary Bridge Plan and supporting information. Supporting information includes all bridge/structures related environmental concerns and suggested mitigation measures, studies of bridge types and span arrangements, approach bridge span layout plans and profile sheets, controlling vertical and horizontal clearance requirements, roadway geometry, design specifications used, special design criteria, special provisions (if available), and cost estimates. In addition, submit hydraulic and scour design studies/reports which show scour predictions and related mitigation measures. Also submit geotechnical studies/reports along with information on substructure and foundation types.

For unusual or complex bridge projects, the State Bridge Engineer will submit one copy of the Preliminary Bridge Plan along with a transmittal letter requesting approval directly to the FHWA Division Bridge Engineer. The transmittal letter also includes the estimated contract construction cost of the structure. The FHWA is the only outside agency to which the Bridge Office sends a direct request for approval. All other outside agencies are contacted through other offices of MnDOT.

Note that the FHWA Headquarters Bridge Division is available for technical assistance on other Federal-aid and non-Federal-aid highways when requested.

- 3) The Preliminary Bridge Plan is used as a basis for preparing permit drawings to accompany applications to construct structures and approaches over navigable waters of the United States within or bordering our state. Such drawings are prepared in the Preliminary Plans Unit in accordance with detailed instructions issued by the U.S. Coast Guard. The Coast Guard is charged with the responsibility of issuing permits for bridges over navigable waters of the United States within or bordering our state. This includes all bridge spans (including land spans) from abutment to abutment. The Corps of Engineers is responsible for issuing permits for any other miscellaneous structures or work to be performed in navigable waters of the United States.

There are two Coast Guard districts that have jurisdiction within the State of Minnesota; the 9<sup>th</sup> Coast Guard District based in Cleveland has jurisdiction over the Duluth harbor and navigable portion of the St. Louis River, and the 8<sup>th</sup> Coast Guard District based in St. Louis has jurisdiction over the navigable portions of the Mississippi, Minnesota, and St. Croix Rivers.

After receiving a permit application, the Coast Guard issues a public notice of application with prints of the permit drawings. These are sent to shipping interests, other agencies, displayed in post offices, etc. Generally, if no comments are received from others within 30 days of the notice of application, and if environmental statements have been submitted and a certification given by the Minnesota Pollution Control Agency, a permit will be issued.

Correspondence to the Coast Guard is generally prepared for the signature of the State Bridge Engineer.

- 4) When all approvals have been obtained, the Preliminary Bridge Plan is used as the basis for the bridge design and for the preparation of final detailed plans. If the design is to be by a consulting engineer, the Preliminary Bridge Plan is typically used as the basis for negotiation of the consultant fee.

Table 2.3.1.2 General Distribution of Preliminary Bridge Plans

DISTRIBUTION TO	PURPOSE		REMARKS
	INFO. & REVIEW	PRE-APPROVAL REQUIRED	
MnDOT District Project Manager	x		
District Pre-Design	x		
District Final Design	x		
District Construction	x		
District Environmental Coordinator	x		
District Hydraulics Engineer	x		For bridges that cross waterways.
District Maintenance	x		
District Bridge Engineer	x		
District Traffic Engineer	x		Send with request for determination of need for lights, signals, conduit, and bridge mounted signs.
Office of Materials & Road Research – Foundations Unit	x		
Regional Transportation Management Center	x		Send with request for determination of need for conduit and mounting devices for surveillance system.
Environmental Stewardship Office	x		
MnDOT Office of Freight and Commercial Vehicle Operations (OFCVO)		x	For railroad crossings only.
Federal Highway Administration (FHWA)	x	x	Approval required for unusual or complex bridge projects only. For all other bridges, a courtesy copy is provided.
Bridge Final Design Unit	x		
Bridge Estimating Unit	x		
Bridge Waterway Unit	x		For bridges that cross waterways.
Bridge Consultant Agreements Unit	x		For bridge projects with consultant involvement.
Bridge Consultant	x		For bridge projects with consultant involvement.
Other Stakeholders	x		As needed.

**Preliminary Plans for Local Bridges**

Consult the State Aid Bridge Web site at:

<http://www.dot.state.mn.us/stateaid/bridge/resources.html> for the submittal and acceptance process of State Aid Preliminary Bridge Plans.

**2.3.2 Bridge Type Selection****General**

The type of structure and span arrangement selected will depend on cost, depth available, geometrics, site conditions, and aesthetics. For some bridges this may be an obvious choice. For others it may involve a great deal of study, especially if aesthetics is a main concern. The section that follows gives some general guidelines on the selection process.

**Aesthetic Design Process**

See Section 2.2 of this manual for a general discussion of the aesthetic design process.

**Structure Type**

The most commonly used structure types and their characteristics are as follows:

**1) Precast Pretensioned Concrete Beam**

This is the most common structure type in Minnesota. Advantages include: low initial and future maintenance costs, high quality factory produced product, a stiff deck, and simple spans that accommodate tapers. Beams are limited to standard depths and straight segments, and a maximum length of about 200 feet. Beams in excess of 150 feet may require special shipping considerations.

**2) Welded or Rolled Steel Beam**

This type of structure is well suited to complex urban freeways with limited depth, long spans, and complex geometrics. Steel beam bridges are also well suited for areas with bad soils, such as the Red River Valley, as steel allows the flexibility of modifying the bearing location and adding or reducing span lengths to accommodate shifting abutments and piers. Advantages include: a shallower depth of structure than prestressed concrete, beams with the ability to be field spliced to produce long span lengths, web plates that can be cut to any depth or to a haunched shape, beams that can be curved horizontally, and beams that can be painted a color which contrasts with the slab to make the structure appear thinner. Disadvantages include: a typically higher cost than other structure types, more difficult fabrication and inspection, a longer fabrication time, the possible need for initial painting and future maintenance painting, weathering steel staining of supports, and rusting of weathering steel when under salt exposure.

### 3) Cast-In-Place Concrete Slab Span

This type of structure is used for shorter span bridges where depth is a major consideration. For simple spans conventionally reinforced, spans range up to 40 feet. Continuous spans are limited to about 60 feet. (See table in Section 5.3.1 of this manual for limits.) Advantages include: a minimum depth superstructure, ease of design and detailing, pleasing aesthetics, and economy for short span bridges. Disadvantages include: span lengths are limited, falsework is required, concrete delivery rate requirements may be a problem, a wearing course may be required to achieve a smooth ride, and the maximum skew angle is 45°.

### 4) Post-tensioned Concrete Box Girder

Concrete box girders provide an attractive structure with high torsional resistance making them especially well suited for curved structures. The ability to accommodate an integral pier cap is an advantage since horizontal clearance is only required to the column top and not the cap top. Limitations and drawbacks may include the need for falsework, the inability to redeck or widen, and the higher construction cost.

### 5) Timber

This bridge structure is used only on the local road system, for 1 or 3 spans with a maximum span length of about 25 feet. Advantages include: timber has a natural and aesthetically pleasing appearance, special equipment is not required for installation, and construction can be done in virtually any weather conditions. Disadvantages include: timber is not an economical structure type, it is limited to low-volume roads (roads with an AADT under 750), and the asphalt wearing surface tends to crack due to differential deck deflections.

### 6) Precast Concrete Box Culvert

Box culverts provide a quickly constructed and economical structure for stream crossings and pedestrian tunnels. Precast concrete box culvert standards are available for culverts up to 16 ft. x 12 ft. in size. Use of up to three large barrel boxes may be economical compared with a bridge. Advantages include: standardized plans, quick installation and low maintenance. Disadvantages include: span limitations, possible debris build-up when multiple barrels are used, and lack of a natural stream for fish unless the invert is lowered and riprapped.

### 7) Three-Sided Bridge Structure

Three-sided precast concrete structures offer an alternative for short span structures up to 42 feet. Advantages include: quick installation, and a natural stream bottom if scour protection is not required.

Disadvantages include: a higher cost than cast-in-place structures, and pile foundations are typically required for stream crossings unless founded on rock.

Not all bridge sites lend themselves to the use of the more common bridge types listed above. For these situations, specialized bridge types may be required, such as post-tensioned I-girder bridges, tied arch bridges, cable-stayed bridges, or extradosed bridges.

### **Abutment and Pier Locations**

The following guidelines aid in setting abutment and pier locations:

#### 1) Water Crossings

For water crossings, keep the number of substructures located in the water to the minimum practical. Piers in rivers and streams block the natural flow of the waterway, trap ice and debris, impede navigation, and are subject to scour. In addition, construction of a pier in the water is expensive (especially if cofferdams are needed), and environmentally disturbs the stream/river/lake bottom and water quality. Ideally, set piers and abutments on shore to minimize dewatering and allow easy access for the Contractor. Set substructures to avoid interference with in-place substructures, including piling, wherever practical. Setting spans and structure depth involves balancing the hydraulic requirements of the low member elevation and waterway area with the constraints of approach grades, structure depth, and cost.

#### 2) Grade Separations

For grade separations fewer piers are also desirable wherever practical. Keep piers out of the clear zone unless absolutely necessary. In locations where ramps enter or exit a highway under a bridge, avoid piers between the mainline and ramp, if possible, as they restrict visibility.

When piers must be located in the median and within the clear zone, place the pier so it is equidistant to the inside edge of traffic lanes in both directions. This will maximize the buffer between the traffic and the pier.

### **Abutment Types**

Abutments can generally be classified into 3 categories: stub, semi-high, and high abutments. A further breakdown of abutments can be made according to the way expansion is handled – integral, semi-integral, or parapet type.

- 1) **Stub Abutment:** This is the shortest category of abutment, located at the top of the fill slope with generally 2 to 4 feet of stem exposure.

Integral type stub abutments are the preferred type of abutment due to their jointless nature and simplified construction. Integral type stub abutments have the lowest initial construction cost, are the fastest abutment type to construct, and eliminate the future maintenance and repair required for strip seal expansion joints. Refer to BDM Article 11.1 for length, skew, and exposure limits for integral type stub abutments.

Semi-integral type stub abutments are the preferred type of abutment when the requirements for integral abutments cannot be met. Semi-integral abutments have a lower initial construction cost than parapet type abutments, and eliminate the future maintenance and repair required for strip seal expansion joints. Refer to BDM Article 11.1 for length and skew limits for semi-integral type abutments.

Parapet type stub abutments use a strip seal or modular expansion device to accommodate movement. They have the highest initial construction cost, and will require future maintenance and repair for the strip seal expansion joints. The move toward jointless abutments has diminished the use of parapet type stub abutments, but this type is still used where appropriate.

- 2) **Semi-high abutment:** This abutment type is located part way up the fill slope and became more popular as two-span overpasses came into use. A slightly higher abutment and elimination of the berm reduces the span length and depth of beam. This allows a lesser profile grade increase, resulting in lower grading costs. Limit the exposed height of abutment face to approximately 8 feet, if possible. Undertake a cost evaluation of longer spans vs. taller abutments when considering a semi-high abutment.

This category includes semi-integral and parapet type abutments only (integral abutment height restrictions limit them to the stub abutment category). Semi-integral type abutments are the preferred type because of their lower initial construction cost and lower maintenance requirements.

- 3) **High abutment:** This abutment type is located at the bottom of the fill slope and is used primarily in congested urban design where structure depth is difficult to obtain. Their use is discouraged since they are more difficult to construct, expensive, require lengthy retaining walls and

approach panels, and give a closed-in feel to the highway. Again, this category includes semi-integral and parapet type abutments only, with the semi-integral type preferred due to its lower initial construction cost and lower maintenance requirements.

In locations where a high abutment would be required and use of a mechanically stabilized earth (MSE) retaining wall is economical, another option is a parapet type abutment supported by a pile foundation behind an MSE retaining wall.

Things to consider when deciding on what height of abutment will best serve a specific project include:

- Advantages of choosing a shorter abutment over a taller abutment:
  - Lower abutment cost.
  - Longer bridge length results in reduced grading and pavement cost.
  - Shorter wingwall and approach panel lengths.
  - Construction of abutments farther from roadway underneath allows for construction staging, possible future expansion underneath for roadway widening or addition of sidewalks, shared-use trails, or other facilities.
- Disadvantages of choosing a shorter abutment over a taller abutment:
  - Requires a longer bridge length, resulting in a higher superstructure cost and increased future maintenance cost.
  - May require a grade raise, resulting in a higher roadway grading and pavement cost.
  - Higher cost for slope protection.

Wingwalls parallel to the bridge roadway are used most often for aesthetic reasons. Flared wingwalls, typically with a flare angle of 45 degrees for bridges with no skew, will result in shorter wingwall lengths and less length of railing. Straight wingwalls, an extension of the abutment parapet, are the simplest to construct but are appropriate only for shallow beams where aesthetics is not a concern.

See additional limitations and guidance for integral, semi-integral, and parapet abutments in Section 11 of this manual.

### **Pier Types**

There is a wide variety of pier types used in bridges, with the most typical consisting of a pier cap supported on multiple columns.

Consider the following general guidelines in order to achieve cost effective piers:

- Minimize the number and size of the columns.
- Minimize the pier width.
- Minimize the number of construction joints in the columns and the pier cap.
- For ease of reinforcement detailing, avoid use of inside corners in the pier column/shaft cross-section.
- For column and cap type piers that require a crash strut, consider use of a solid shaft type pier.

The feature being crossed is an important consideration when choosing the pier type. Discussions on pier type are provided below for water crossings and grade separations.

#### 1) Water Crossings

**Pile Bent Piers:** These piers consist of a row of piles with a concrete cap encasing the pile top, and are the simplest and most economical type of pier. They are used for water crossings where a general maximum height from the top of pier to stream/river/lake bed is under 20'-0" and there is no ice or debris problem. Note that it is important to confirm by analysis that the pile unbraced length under a scour condition does not create instability in the pile. Spans must also be short enough to allow a single row of piles to support the deck at reasonable spacing. The piles act as columns, and bending strength to resist side impacts from ice or debris is important. For cast-in-place piles (the most widely used), a 16" minimum diameter is required. If H-piles are used, the upper portion is encased by a cast-in-place pile shell filled with concrete. Timber piles are not permitted. Concerns with pile bent piers include the potential to trap debris, pile stability, and appearance.

**Wall Type Piers:** These piers consist of a single row of piles (usually H-piles) encased with concrete to form a wall. They provide more resistance to ice and debris and allow debris to pass through without becoming entangled on the piles. This type of pier is used where more resistance to ice and debris than afforded by the pile bent is needed, and yet the size and expense of a solid shaft pier can be avoided. This type of pier can be constructed by driving the piling, supporting the wall forms on the stream/river/lake bed, placing a seal with a tremie, dewatering, adding reinforcement, and pouring the wall. Pile stability can be a concern and must be evaluated.

**Solid Shaft or Multiple Column Piers:** These piers are used for major water crossings where tall piers are required or where heavy loads or

sizable ice and debris loads may occur. This type of pier includes a footing with the bottom of footing located a minimum of 6'-0" below the stream/river/lake bed. Construction of this type of pier involves driving sheeting to form a cofferdam, excavating inside the cofferdam, driving piles, pouring a seal, dewatering, and placing concrete.

## 2) Grade Separations

Piers at grade separations are typically multiple column type with a cap. Piers are visible to passing motorists and the emphasis on aesthetics has led to more use of rectangular shaped column type piers, often with form liner treatments or rustication grooves. For narrow ramp bridges, a single shaft pier may be considered. Where aesthetics is not a concern, a round column pier will usually provide the lowest cost.

For the majority of bridges over roadways, piers located within 30 feet of the roadway edge (defined as the edge of the lane nearest to the pier) must be designed to withstand a 600 kip load unless they are protected as specified in LRFD 3.6.5.1. This may impact the aesthetics by requiring inclusion of a crash strut. The alternative is to provide columns with a substantial cross-section designed to resist the crash load or protect them with a TL-5 barrier. See Article 11.2.3 of this manual for complete pier protection policy and requirements.

For bridges over railroads, piers located within 25 feet of the centerline of railroad tracks must either be of "heavy construction" or have crash walls. Refer to Article 11.2.3.2.2 of this manual for complete requirements.

See Section 11 of this manual for additional guidance on piers.

### ***2.4 Final Bridge Plans and Special Provisions***

The primary purpose for preparing the Final Bridge Plan and special provisions is to communicate the geometric, material, and procedural requirements for the construction of a bridge. Several audiences will use the final bridge plan or contract documents during the life of the bridge. Initially, contractors use the documents to prepare their bids. A clear, accurate, and complete set of documents will result in competitive bidding. Well-communicated information reduces contractor uncertainty regarding what is required for different elements of construction.

During construction, many parties will use the contract documents. For example, surveyors will locate and mark the position of working points, fabricators and construction engineers will prepare shop drawings and

other submittals/drawings, inspectors and suppliers will use the documents for their work, and the contractor's forces will use the documents.

After construction of the bridge the detailed plans will be referenced when modifying the bridge (e.g., adding signage), performing load rating of the bridge, or rehabilitating/replacing the bridge.

The Final Bridge Plan contains geometric information, a schedule of quantities and pay items for the bridge, traffic phasing (if applicable), limits of removal of existing structures and foundation items (if applicable), foundation details, substructure details, superstructure details, typical sections, utilities (if applicable), survey information, and other miscellaneous items.

Specifications are also required for each project. They describe procedures for award and execution of the contract, how work will be measured and paid, procedures to be followed during execution of the work, and material and testing requirements for items incorporated into the project.

Bridge projects use specifications from four different sources:

- 1) Most of the specifications used for a project are provided in MnDOT's *Standard Specifications for Construction*. They are necessarily general in nature and are intended to cover all types of MnDOT projects.
- 2) The Bridge Office has assembled additional specifications. Because they are not included in the standard specifications they are called special provisions. A list of available standard bridge special provisions (*2018 "SB" Bridge Special Provisions*) is provided on the Bridge Office web site at: <http://www.dot.state.mn.us/bridge/construction.html>. Special provisions address a variety of work items, ranging from concrete placement to the fabrication and installation of expansion joint devices. Not all of the special provisions are intended to be used on every project; use only those applicable to the project.
- 3) The Bridge State Aid Unit has additional standard bridge special provisions that apply to local road bridge projects.
- 4) Custom special provisions. If a work item is of such unique character that the standard specifications and the standard bridge special provisions don't describe or address the work, a custom special provision will need to be prepared. Custom special provisions may be generated for any number of items. Items may include schedules (e.g., dates the contractor will have access to certain portions of the project) or lists of required submittals, etc.

In general, information that is highly graphical or geometric in nature should be presented on plan sheets. Large amounts of information conveyed with text should be assembled in special provisions.

A specification or special provision usually contains the following five sections:

- 1) Description of work
- 2) List of the materials used (and their specifications)
- 3) Construction requirements for the work
- 4) Description of how the work will be measured
- 5) Basis of payment (pay item for the work)

#### ***2.4.1 Final Design Instructions***

Unless specified otherwise within this manual, design all structures in accordance with the current AASHTO *LRFD Bridge Design Specifications*. For those few cases where LRFD specifications have not been created or adopted, discuss options with the State Bridge Design Engineer prior to beginning final design.

Design railroad bridges according to the current AREMA specifications for the live load specified by the railroad. Additional notes concerning the design of railroad bridges:

- 1) Railroad bridges will usually be designed with simple spans to avoid uplift from the live load.
- 2) Bridges for the Duluth Mesabe & Iron Range Railway require a special live load.

Plans and documents prepared during the preliminary design phase should be reviewed prior to beginning final design. These documents include:

- 1) Preliminary Bridge Plan
- 2) Bridge Construction Unit Foundation Recommendation Report
- 3) Design Study Report (if completed)
- 4) Preliminary Design Folder (found in ProjectWise)

When reviewing preliminary plans, pay particular attention to geometry and utilities. Check the layout. This includes reviewing grades, stationing, end slopes, beams, railings, roadways, shoulders, and the median (if applicable).

#### ***2.4.1.1 Superstructure***

Space beams so moments in fascia beams will not be larger than moments in interior beams.

**2.4.1.1.1 Framing  
Plan**

For steel beams and pretensioned I-beams, deck projections beyond the centerline of the fascia beam should generally not exceed the smallest of:

- 1) Depth of beam: During construction, overhang support brackets that support deck forms, safety walkway, etc., contain a diagonal member that is supported off the beam bottom flange. If the overhang exceeds the beam depth, analyze to check if bracing of the beam is required. Include a note in the bridge plan if bracing is required.
- 2) 40% of the beam spacing: This limit keeps the deck overhang moment and the exterior beam dead load within a reasonable range. If exceeded, Section 9 deck tables cannot be used and a special design is required.
- 3) Deck coping width + barrier width + 1'-0" + ½ flange width: This keeps the design truck wheel within the limits of the exterior beam top flange, thereby ensuring that the live load will not govern the deck overhang design.

For rectangular pretensioned beams, deck projections beyond the centerline of the fascia beam should generally not exceed the smaller of:

- 1) 40% of the beam spacing: This limit keeps the deck overhang moment and the exterior beam dead load within a reasonable range. If exceeded, Section 9 deck tables cannot be used and a special design is required.
- 2) Deck coping width + barrier width + 1'-0" + ½ beam width: This keeps the design truck wheel within the limits of the exterior beam top flange, thereby ensuring that the live load will not govern the deck overhang design.

Provide a minimum slab projection beyond the tip of the flange of 6 inches.

**2.4.1.1.2 Bridge  
Decks and Slabs**

For bridges with reinforced concrete decks or slabs, the deck or slab may be cast in one lift (monolithic) or two lifts (deck/slab plus low slump wearing course). Note that the wearing course and the future wearing course are separate and distinct items.

**Bridge Deck Protection Policy**

For new bridge decks and slab span superstructures, utilize:

- High Performance Concrete (3YHPC). In remote areas of the state where ready mix suppliers cannot produce 3YHPC, use Low Cracking High Performance Concrete (3YLCHPC).
- Monolithic Deck or Slab (no separate wearing course).
- Synthetic Fibers (a combination of micro and macro synthetic fibers).

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A list of common conditions that may warrant exceptions to the above is provided in Table 2.4.1.1.2.1. Note that these exceptions apply only to bridges with AADT greater than 2,000.

For new bridges in remote areas and bridges that meet any of the conditions found in Table 2.4.1.1.2.1, the Preliminary Bridge Plans Engineer will consult with the Regional Bridge Construction Engineer and the District to determine the appropriate concrete mix and whether a wearing course is required. The concrete mix type and type of deck or slab (monolithic deck/slab or deck/slab with wearing course) will be included in the Preliminary Bridge Plan.

For existing bridge decks and bridge slab span superstructures, refer to the MnDOT Bridge Preservation and Improvement Guidelines (BPIG) for guidance.

**Table 2.4.1.1.2.1**

**Bridge Conditions That Require Consideration of a Low Slump Wearing Course ①**

<u>Condition</u>	<u>Commentary</u>
1) Project locations where HPC concrete is not available.	Not all MN concrete plants have successful history with HPC concrete production and delivery.
2) Bridge is located on a constant grade < 0.83%.	Variations in superstructure deflections and finishing tolerances can make positive drainage difficult.
3) Bridge has a continuous steel superstructure with degree of curvature > 10 degrees.	Behavior of steel superstructure deflections and rotation during sequential pouring can be difficult to adequately predict to achieve ride tolerance.
4) Skew > 30 degrees on 2 spans or more with an aspect ratio (deck width/span length) > 0.5.	Finishing rails must deflect uniformly to produce the most uniform cross-section. Finishing of skewed bridges is best accomplished by placing wet concrete uniformly on all beams within the span by setting the finishing machine on a similar skew to substructures.
5) Bridge is located on a vertical curve with approach grades > 3% and support skews > 20 degrees.	Where a vertical profile and skew exists, the difference in elevation at either rail may produce a warped superelevation if finished on skew.
6) Superelevation transition occurs on the bridge.	Finishing machines cannot easily accommodate variable superelevation breaks during a pour.
7) Bridge deck or slab has a longitudinal construction joint due to traffic staging or large deck width.	Multiple pour placements with longitudinal construction joints are more prone to cracking during deflections incurred during the casting sequence. A concrete wearing course placed after major deflections have occurred results in better crack size control.
8) Variable width bridges such as single-point interchanges that are difficult to finish with a finishing machine.	Finishing machines have limited ability to expand width, and finishing outside of the screed rail locations requires finishing equipment with higher risk of placement irregularities.

① Applies only to bridges with AADT greater than 2,000.

**2.4.1.1.3  
Diaphragms and  
Cross Frames**

For most bridges, the orientation of the primary superstructure elements is parallel to the centerline of the bridge. Aside from slab bridges, most bridges in Minnesota are supported on multiple beam lines. The beam lines are typically spaced on 5 to 15 foot centers. These bridges usually have diaphragms or cross frames, which serve a number of purposes:

- 1) They provide compression flange bracing during erection and construction of the bridge.

- 2) They increase lateral load distribution (more beams or girders participate in carrying live loads).
- 3) They provide a load path for lateral loads to be carried from the deck to the bearings.

During final plan assembly, specify the type of diaphragm on the framing plan, the deck cross section, and the longitudinal section.

For bridges with integral or semi-integral abutments, the end diaphragm also functions as an abutment element.

#### **2.4.1.2 Pedestrian Bridges**

Pedestrian bridges shall be designed in accordance with the *Guide Specifications for Design of Pedestrian Bridges*. Several additional constraints are placed on pedestrian bridges to ensure they are accessible, safe, and durable:

- 1) For guidance regarding determination of pedestrian bridge width, refer to Article 2.1.2 of this manual under **Shared-Use Paths and Pedestrian Walkways on Bridges**.
- 2) The maximum grade permitted on a pedestrian bridge is 8.33%. A grade flatter than the maximum is preferable. When the grade equals or exceeds 5%, provide a 5'-0" platform for each change in elevation of 2'-6". Also, a handrail is required when the grade equals or exceeds 5% per ADA requirements.
- 3) Protective screening, preferably a chain link fence system or a railing system, must be placed on both sides of the bridge. The height of the fence or railing must be 8'-0" above the top of the sidewalk. For sites with special aesthetic treatments involving ornamental railings, a minimum height of 6'-0" will be allowed. Where a fence or railing system is also required on retaining walls that are connected to the bridge, it is recommended that the same system (chain link fence or railing) be utilized throughout.
- 4) Provide a 6'-0" clear platform at the bottom of each ramp.
- 5) Provide a platform at each abrupt change in a horizontal direction. The minimum plan dimension for a platform is 5'-0" by 5'-0".
- 6) Lay out the profile grade such that there are no abrupt grade breaks at expansion devices.
- 7) Only in the rare case where handicap accessibility need not be provided can stairs be incorporated into a design. When stairs are provided, use the following guidelines:
  - a) Provide stairs with a 1'-0" tread and a 6" rise.
  - b) Adjust the sidewalk or superstructure elevations to make all risers 6" tall.

- c) The preferred number of risers in a flight of stairs is 14 to 16. The maximum number is 19.
- 8) Detail the rails in accordance with the following:
  - a) Refer to Section 13 of this manual for metal railing height and spindle spacing requirements.
  - b) When required, place handrails 2'-8" above the top of the deck.
- 9) Provide an electrical ground for continuous chain link fences, ornamental railings, and metal handrails. If appropriate, provide bicycle ramps on pedestrian bridges that contain stairs.

### **Materials**

Use steel, prestressed concrete, reinforced concrete, or timber for the superstructure of pedestrian bridges. Aluminum is not an acceptable material for use in any portion of the superstructure.

The minimum structural steel thickness is  $\frac{1}{4}$  inch for pipe or tube sections and  $\frac{5}{16}$  inch for all other sections. The minimum thickness requirements do not apply to railings. Provide structural tubing details that are watertight or designed such that moisture cannot be trapped in or on the member to accelerate corrosion.

Use a high performance concrete mix for the deck of a pedestrian bridge. Contact the Regional Bridge Construction Engineer to determine which mix is most appropriate for the site: 3YHPC-M or 3YLCHPC-M.

The Brazilian hardwood known as IPE, though very durable, is not an accepted decking material on state or federally funded projects. If the use of IPE wood is desired by the owner, local funds are the only option for payment.

### **Bridge Substructure**

Use reinforced concrete supported on piling, drilled shafts, or spread footings for bridge substructures as recommended in the Bridge Construction Unit Foundation Recommendations report. Incorporate drainage systems (Detail B910) into the abutments as needed.

### **Bridge Superstructure**

To limit transverse deck cracking due to negative flexure, provide additional longitudinal bars in the top of the deck over the piers. Stagger the ends of the additional longitudinal bars to transition the capacity of the section. (See Figures 9.2.1.8 and 9.2.1.9.)

Detail anchorages for the piers and abutments to resist uplift and overturning forces associated with wind loads.

Provide a cover plate over all pedestrian bridge expansion joint openings to protect pedestrians from a tripping hazard.

Type 5.0 strip seals with expansion joint openings up to 5.0 inches are allowed on pedestrian bridges since the joint is concealed by a cover plate.

#### **Highway Geometrics**

Meet MnDOT design standards for horizontal and vertical clearances for a pedestrian bridge over a roadway.

#### **2.4.1.3 Temporary Bridges and Widening**

##### **Temporary Bridges**

Temporary bridges are used to detour traffic while removal of an existing bridge and construction of a new bridge occur on the mainline of the roadway.

Design temporary bridges in accordance with the LRFD Specifications using the HL-93 live load with an associated load factor of 1.75.

For posted speeds in work zones of 40 mph or less, design the barriers, the barrier/deck connection, and the deck overhang to meet railing Test Level 2. For speeds greater than 40 mph, design to meet Test Level 3.

##### **Temporary Widening**

Temporary widening occurs when staging requires widening of an existing bridge while construction of an adjacent new bridge occurs.

Design structural components of the temporary widening to meet or exceed the capacity of the existing bridge components. For a temporary widening, match the deck material of the existing bridge.

For temporary widening projects, design the barriers, the barrier/deck connection, and the deck overhang to meet the barrier test level required for the roadway.

#### **2.4.1.4 Bridge Approaches**

In most cases, the bridge approach panel will be included with the roadway grading plans for a project. Guidance for the treatment and details of approach panels can be found in the following:

##### **Bridge Approach Treatment**

The approach treatment standard sheets describe the limits and treatment of excavation and backfill near the abutments. These sheets are found in the *MnDOT Standard Plans Manual*, Figures 5-297.233 and

5-297.234. The Preliminary Bridge Plan contains a note indicating which approach treatment sheet to use.

#### Bridge Approach Panel

The standard sheets covering bridge approach panels are found in the *MnDOT Standard Plans Manual*, Figures 5-297.222 through 5-297.231. These figures cover standard approach panels for abutments with joints, abutments without joints, abutments with different amounts of skew, different mainline pavement types, and miscellaneous details. The Bridge Preliminary Plan contains a note indicating which approach panel sheets to use.

Specify a concrete wearing course on approach panels when the bridge deck has a concrete wearing course. The wearing course on the approach panels will be placed at the same time as the wearing course on the bridge. Include the approach panel wearing course quantity in the summary of quantities for the superstructure.

#### **2.4.1.5 Survey**

When assembling the survey sheets for final plans, verify that the most current grading plans are being used.

Include the centerlines and object lines for the abutment and pier footings on the final design survey sheets. Also identify and locate all test piles.

#### **2.4.1.6 Utilities**

The Bridge Office Preliminary Plans Unit in coordination with the District Traffic Engineer determines if provisions must be made for safety lighting (roadway, navigation, inspection, etc.), signing, or signals. Coordination is also done with the MnDOT District Project Manager regarding the need for other types of utilities.

#### **2.4.1.6.1 Suspended Utilities & Utilities Embedded in Bridges**

The conduit for utilities is to be suspended below the deck on hanger systems between the beams. Locate the entire conduit and hanger system above the bottom of the beams and generally below the diaphragms or in the lower openings of a cross frame diaphragm. To minimize the impact to the structure in the future, avoid casting conduits for utility companies in the deck, sidewalk, or barriers/parapets.

Use polyvinyl chloride (PVC) coated hot dipped galvanized rigid steel conduit (RSC) for utilities requiring conduit. Use galvanized steel hangers and supports.

Roadway lighting conduit ( $1\frac{1}{2}$  inch diameter maximum) will be allowed in barriers/parapets (maximum of 2 per barrier/parapet) and sidewalks.

Suspended water, sewer, communications, and electrical power (less than 35kV) utility systems are allowed on bridges. However, natural gas pipelines are considered a safety risk and will not be allowed.

For hanging conduit systems on bridges with parapet or semi-integral type abutments, and when conduit is embedded in concrete barrier, deck, or sidewalk, use a combination expansion/deflection fitting at the abutments. This will accommodate horizontal movements (due to temperature change, creep, shrinkage, etc.) and vertical movements (due to jacking operations for bearing replacement, etc.).

For hanging systems on bridges with integral abutments, only an expansion fitting is required at the abutments.

The temperature movements of RSC approximate those of concrete. Consequently, lateral bracing is not needed. Choose a transverse spacing for the conduits that permits proper placement of concrete between embedded anchors.

Typical conduit and utility details are available in the Bridge Office final design cell library, available at:

<http://www.dot.state.mn.us/bridge/drafting-aids.html>

#### ***2.4.1.6.2 Buried Utilities***

To protect structures, restrictions on the location of new or existing buried utilities and drainage pipes must be considered near bridge substructures and wall structures supported on spread footings, and also near sheet pile walls, soldier pile walls, and mechanically stabilized earth (MSE) walls. Location restrictions, installation techniques, protection measures, and plan review are required for utilities in the utility critical region immediately adjacent to the structure, defined below for the type of structure. Installation in the utility critical region requires review and approval from the MnDOT Bridge Office. Additional restrictions on the location of utilities may be specified in other documents relevant to the project.

For purposes of this section, utilities are defined as any utility requiring a permit as well as State owned utilities and stormwater structures. Dry utilities are defined as facilities that do not carry fluid, such as power and telephone. Wet utilities are defined as those facilities that carry fluid or gasses, but do not include roadway edge drains or subsurface drains associated with the bridge or wall structure.

**Utilities Near Bridge Substructures and Walls Supported on Spread Footings, and Sheet Pile Walls and Soldier Pile Walls**

The limits of the utility critical region are defined as 50 feet lateral to, 50 feet below, and 15 feet above the base of foundations for bridge substructures and walls supported on spread footings. For sheet pile walls and soldier pile walls, the limits of the utility critical region are defined as 50 feet lateral to each edge of pile, 50 feet below the pile tip, and the vertical distance above the pile tip to the ground elevation on the non-retained earth side of the wall.

Within the utility critical region, three zones have been identified to provide general guidance for MnDOT approval. See Figure 2.4.1.6.2.1 for the definition of the zones.

All wet utilities in Zones 1, 2, and 3 require pipe with gaskets or joints designed to prevent leakage due to pressurized flow. Casing, where required, will meet MnDOT requirements for casing. Refer to the *MnDOT Policy Statement on Accommodation of Utilities on Highway Right of Way* for casing requirements.

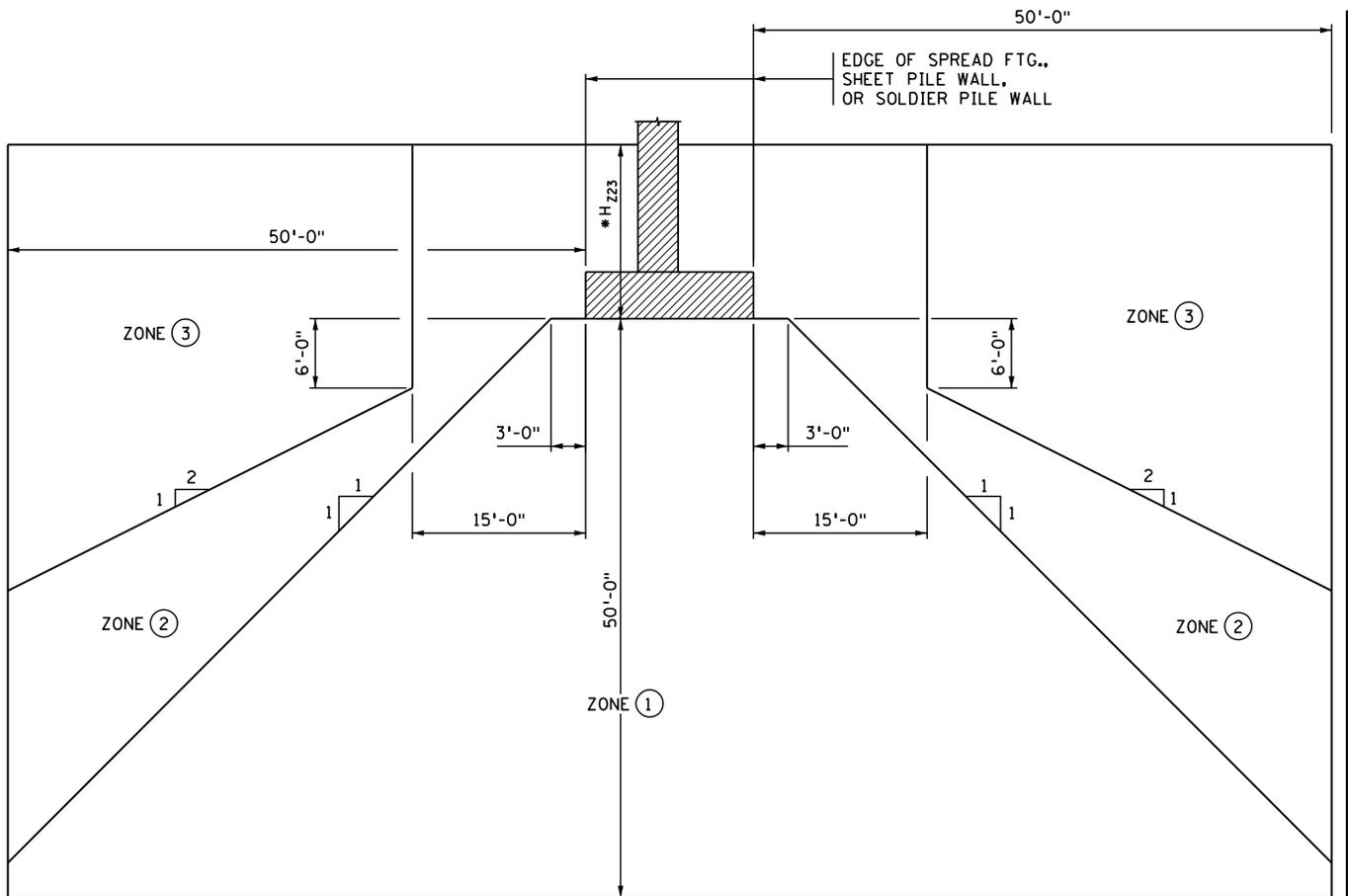
The following restrictions on utility placement are dependent on their position relative to the structure:

**Zone 1**

During the construction of the foundation, utilities are to be placed outside of Zone 1 when possible. If relocation is impractical or impossible, the requirements for locating utilities in Zone 1 are as follows:

- New utilities to be installed and existing utilities to remain in place require Bridge Office approval.
- New wet or dry utilities may be placed transversely (i.e., perpendicular to the substructure or wall) to an existing structure in Zone 1, with Bridge Office approval of proposed design and construction sequencing.
- No new wet utilities may be placed longitudinally (i.e., parallel to the substructure or wall) in Zone 1. Dry utilities are allowed longitudinally.
- All pipes and conduits must be designed for any surcharge loading due to structure bearing pressures and possible resulting deformations.
- All wet utilities must be cased in Zone 1; if facilities are too large or cannot be cased effectively, a site specific design is required.

- Utility owners may choose to case dry utilities to allow for future maintenance or access; however, casing is not required for dry utilities.
- Future open trench excavation is prohibited in order to protect the substructure or wall from potential undermining. Other forms of excavation may be permitted in this zone with Bridge Office approval.



\* FOR BRIDGE STRUCTURES AND WALLS SUPPORTED ON SPREAD FOOTINGS:  
 $H_{z23} = 15'-0''$ .  
FOR SHEET PILE WALLS AND SOLDIER PILE WALLS:  
 $H_{z23}$  = DISTANCE FROM PILE TIP TO GROUND ELEVATION ON  
NON-RETAINED EARTH SIDE OF WALL.

**Figure 2.4.1.6.2.1**  
**Utilities Near Bridge Substructures and Walls**  
**Supported on Spread Footings, and Sheet Pile Walls**  
**and Soldier Pile Walls Foundations**

### Zone 2

The requirements for locating utilities in Zone 2 are as follows:

- New wet or dry utilities may be installed in Zone 2.
- Excavation for maintenance or replacement will be permitted with proper sheeting and shoring; no unbraced open cuts will be allowed.
- Any utilities installed in Zone 2 must follow the same casing requirements as in Zone 1, with the exception of stormwater facilities. Casing is required for stormwater pipes with velocities greater than 10 fps, or pipe diameters 54 inches and larger, or pipe materials other than those shown in Standard Plate 3006. Other stormwater facilities need not be cased in Zone 2 unless required by contract specifications or as recommended by the Bridge Office.

### Zone 3

There are no restrictions for utility installations in Zone 3 except for the requirement to use pipe with gaskets as needed for wet utilities.

If the conditions above cannot be met, options include relocation or replacement of the utility or placing the substructure on deep foundations (piles or drilled shafts). However, pressurized wet utilities and large, high velocity non-pressurized wet utilities placed in Zone 1 of a deep foundation must be cased due to the risk of significant soil loss. In lieu of casing, a risk analysis approved by the Regional Bridge Construction Engineer is acceptable for substructures on deep foundations.

Certain types of utilities may pose a significant risk to shallow foundations when placed in Zones 1, 2, or 3, or to deep foundations when placed in Zone 1. If these types of utilities were to fail, the foundation would be at risk of failure due to the loss of material from localized scour or erosion. The determination of high risk utilities/high risk foundations will be made on a case by case basis by the Bridge Office and will be based on many factors including, but not limited to utility location, flow pressure, flow rate, structure size, and utility size. Additional restrictions to the ones contained in this document could be applied to utilities that pose a significant risk to the foundations.

### **Utilities Near MSE Walls**

For MSE walls, the limits of the utility critical region are defined as 50 feet lateral to the front of the wall, 50 feet measured laterally beyond the end of the soil reinforcement, 50 feet below the bottom of the leveling pad, the top of the fill on the front side of the wall, and the top of the reinforced earth/retained fill in the back of the wall. Refer to Figure 2.4.1.6.2.2.

Five zones have been identified to provide general guidance for MnDOT approval when utilities are near MSE walls constructed using earth reinforcement. The zones are defined in Figure 2.4.1.6.2.2.

All wet utilities in Zones 1 through 5 require pipe with gaskets or joints designed to prevent leakage due to pressurized flow. Casing, where required, will meet MnDOT requirements for casing. Refer to the *MnDOT Policy Statement on Accommodation of Utilities on Highway Right of Way* for casing requirements.

The restrictions on utility placement are dependent on their position relative to the structure:

#### Zones 1, 2, and 3

Zones 1, 2, and 3 restrictions for MSE walls are the same as for bridge substructures. See restrictions given previously for these zones found under **Utilities Near Bridge Substructures and Walls Supported on Spread Footings and Sheet Pile Walls and Soldier Pile Walls**.

#### Zone 4

The requirements for locating utilities in Zone 4 are as follows:

- New wet or dry utilities may be installed in Zone 4.
- Casing is not required in Zone 4.
- When excavating within this zone, install shoring or sheet piling (,without damaging the geomembrane,) to protect the reinforced zone.
- Excavation for maintenance or replacement will be permitted with proper sheeting and shoring; no unbraced open cuts will be allowed.

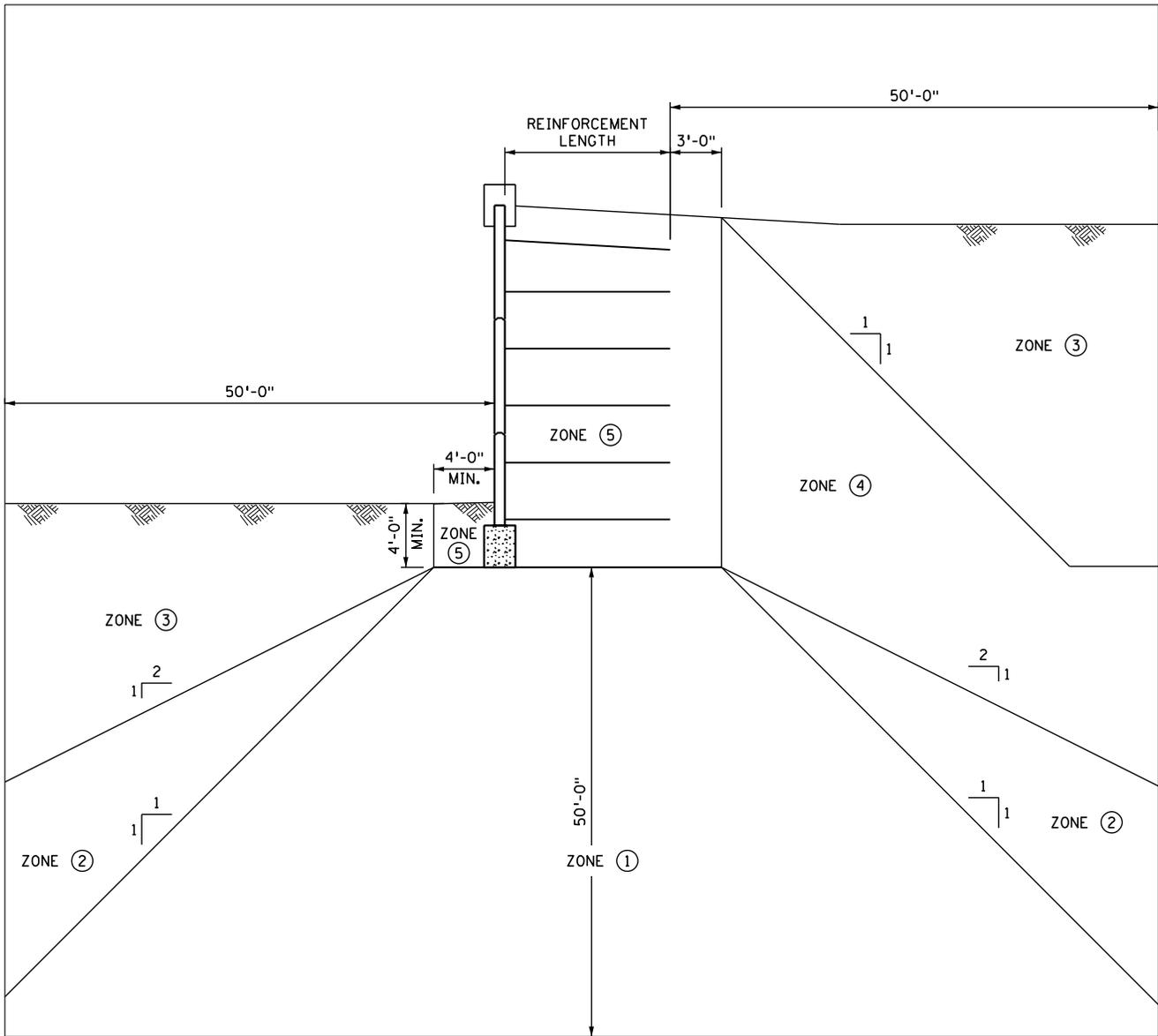
#### Zone 5

Utilities are to be placed outside of Zone 5 when possible. If relocation is impractical or impossible, the requirements for locating utilities in Zone 5 are as follows:

- Existing utilities cannot remain in place as they will be disturbed during construction.
- New utilities to be installed require Bridge Office approval.
- New wet or dry utilities may be placed transversely (i.e., perpendicular to the MSE wall), but only during the original construction of the wall.
- No new wet utilities may be placed longitudinally (i.e., parallel to the substructure or wall). Dry utilities may be placed longitudinal, but only during the original construction of the wall.
- All wet utilities must be cased in Zone 5; if facilities are too large or cannot be cased effectively, a site specific design is required.

- Utility owners may choose to case dry utilities to allow for future maintenance or access; however, casing is not required for dry utilities.
- Future excavation is prohibited in this zone in order to protect the structural integrity of the MSE wall.

Certain types of utilities may pose a significant risk to shallow foundations when placed in Zones 1 through 5. If these types of utilities were to fail, the foundation would be at risk of failure due to the loss of material from localized scour or erosion. The determination of high risk utilities/high risk foundations will be made on a case by case basis by the Bridge Office and will be based on many factors including, but not limited to utility location, flow pressure, flow rate, structure size, and utility size. Additional restrictions to the ones contained in this document could be applied to utilities that pose a significant risk to the foundations.



**Figure 2.4.1.6.2.2**  
**Utilities Near MSE Walls**

**2.4.1.7 Precedence  
of Construction  
Documents**

Designers, while striving to produce accurate error-free construction documents, may at times end up with documents that have conflicting content. A hierarchy has been established to determine which content is governing for a project. In general, the more project specific the document, the higher the document's position in the hierarchy. Section 1504 of the *Standard Specifications for Construction* describes the precedence of construction documents for a project:

If discrepancies exist between the Contract documents, the following order of precedence applies:

- 1) Addenda
- 2) Special Provisions
- 3) Project-Specific Plan Sheets
- 4) Supplemental Specifications
- 5) Standard Plan Sheets and Standard Plates
- 6) Standard Specifications

If discrepancies exist between dimensions in the Contract documents, the following order of precedence applies:

- 1) Plan dimensions
- 2) Calculated dimensions
- 3) Scaled dimensions

**2.4.1.8 Design  
Calculation  
Requirements**

Office practice is to permit the limit states to be exceeded by a maximum of 3%. However, caution should be exercised to ensure that a 3% exceeded limit state at a particular location does not adversely affect the structure load rating.

**2.4.2 Final Plans**

The plan order shall typically follow this list:

- General Plan and Elevation
- Cross Section and Pay Items
- Staging Plan
- Working Point Layout
- Removal Details
- Abutment Details and Reinforcement
- Pier Details and Reinforcement
- Framing Plan
- Beam Details
- Superstructure Details and Reinforcement
- Plan Details (Railing, Expansion Joint, Slope Paving, Conduit, etc.)
- B-Details
- As-Built Bridge Data

- Surveys
- Borings
- Unrevised Informational Sheets

When presenting geometric information, enough baseline information needs to be provided to permit others to verify the information presented. For example, the top of roadway elevations presented on a bridge layout sheet can be confirmed by others using vertical curve information on the general elevation and the cross slopes provided on the typical transverse section.

In general, do not present the same dimensions several times. Providing dimensions in multiple locations increases the chance that not all dimensions will be updated as changes occur during the design process.

The clarity of the details used in plan sets should be a primary concern of designers. Only the simplest details should combine the presentation of concrete geometry and reinforcement. In most cases there is less confusion if two details are used, one to convey concrete geometry and a second to identify and locate reinforcement.

Show the initials of the individuals responsible for the design, drafting, design check, and drafting check on all plan sheets except the as-built sheet, survey sheets, boring sheets, and unrevised information sheets (such as those showing alignment tabulations, superelevation transitions, or aesthetics) taken from the preliminary bridge plan. Note that these unrevised informational sheets are to be placed at the end of the bridge plan. For the boring sheets, show the initials of the individuals responsible for the drafting and drafting check. Similarly, all sheets, except the as-built sheet, survey sheets, boring sheets, and unrevised information sheets must be certified by a Professional Engineer licensed in the State of Minnesota.

In most cases, details are presented with stationing increasing as one moves from the left side to the right side of the sheet. Always include a north arrow on plan views. Plan views are typically oriented with north arrows pointing toward the top or to the right of the sheet. Stationing increases for northbound and eastbound traffic.

#### **2.4.2.1 Drafting Standards**

The Bridge Office has adopted standards to be used when drafting plan sheets. Download *Summary of Recommended Drafting Standards* from: <http://www.dot.state.mn.us/bridge/drafting-aids.html>

### **2.4.2.2 Drafting Guidelines**

#### **Sheet Layout and Continuity**

Read plans from a contractor's perspective to check that they contain all information needed to build the bridge. Make sure enough dimensions are given for constructability. Use extra details for uncommon work. Use perspective views when clarity is needed.

Use sheets efficiently. Balance the drawings on sheets to avoid one sheet being empty while another is crowded. Use additional sheets, as needed, to avoid crowding details on sheets. Make sure that details, data, and other information given on more than one sheet agree between sheets. Avoid unnecessary repetition of details and notes.

Large-scale corner details are required for all skewed bridges and for other complex corners.

Round dimensions to the nearest  $\frac{1}{8}$  of an inch.

Note and dimension bar splices.

Cross-referencing sheets to details is recommended.

Use bill of reinforcement tables for all but very minor reinforced concrete work. Do not enlarge details (such as rebar bends) just to fill up space. Referencing bar bend details by letter to various generic shapes should never be used.

Keep details together for abutments, piers, superstructure, etc.

For abutments, piers, and other complex drawings, use different views and sections to separate dimensions and reinforcement.

Place pile design loads and notes pertaining to a particular substructure on the sheet that contains the footing plan view.

For bridges with numerous footings and curved alignment, a separate foundation layout drawing is recommended.

If the plan contains numerous variable dimensions and other data (especially for framing plans and beams), make use of tables to keep this data in order.

On the Framing Plan, show bearing type beside each bearing point instead of lines and arrows, which tend to clutter the drawing.

For simple beam spans (prestressed beams, etc.), dimension beam spacing at pier cap along centerline of the pier(s). Include supplemental dimensions along centerline of bearing for curved and flared structures.

On projects with staged construction, use enough drawings to clearly indicate how the bridge construction is to be coordinated with the staging. Keep structure units together. Reinforcement and quantity tabulations are to be split between stages.

On repair projects, clearly indicate cut lines and extent of all removals. If there is a saw cut, be sure to use a straight line (WT=5). If elevations are taken off original plans, note as such and require the contractor to verify elevations in the field.

When it appears that plan notes, such as procedure descriptions, specifications, etc., will become excessively wordy, relegate these notes to the special provisions.

List general notes first and specific numbered notes last. Number specific detail notes with circles and reference the detail to which they apply. Place all notes together on the right hand side of the sheet.

Leave extra lines in the Summary of Quantities and Bill of Reinforcement for additions. Also, leave extra space in the list of notes.

### **Pay Quantities**

Make computations neat and readable. Strive for continuity. These computations may be needed for future reference and the reader must be able to interpret them.

Box in or underline computation totals for quicker take off. Initial, date, and put the bridge (or project) number on every computation sheet.

Two sets of independently worked quantity computations are required for each pay item.

Arrange design and quantity computations into a neat and orderly package.

In addition to pay item quantities, compute informational quantities. Do not include these quantities in the bridge plan, but instead submit them to the Bridge Estimating Unit for use in developing the Engineer's Estimate. Examples of information quantities include: summary quantities for conduit systems, summary quantities for drainage systems, cubic yard quantities for concrete items paid for by the square foot or linear foot, and

miscellaneous incidental items such as polystyrene and waterproofing membrane. Sample forms for reporting informational quantities are available at: <http://www.dot.state.mn.us/bridge/design.html>

#### **Additional Drafting Guidance**

Additional drafting resources can be found at:

<http://www.dot.state.mn.us/bridge/drafting-aids.html>

These resources include:

- *Suggested Reinforcement Detailing Practices*
- *Guidelines for Checking Final Design Bridge Plans*
- Bridge Preliminary and Final Design Seed Files
- Bridge Preliminary and Final Design Cell Libraries

#### **2.4.2.3 General Plan and Elevation**

The General Plan and Elevation sheet is intended to summarize the primary features and horizontal geometry of the bridge. Figure 2.4.2.3.1 shows an example General Plan and Elevation sheet and Figure 2.4.2.3.2 shows a Typical Cross Section sheet with pay items.

##### **Plan**

On the plan view identify the following: working points, working line, centerlines, utilities, location of in-place bridges or substructures, ditch drains, deck drains, lights, and nameplate. Label the following: span lengths, deck width, size of angles between the working line and centerlines, horizontal curves, minimum horizontal clearance to substructure units, point of minimum vertical clearance for each roadway under the bridge, extent of slope protection, roadway stationing and elevations, and distance between twin bridges. Provide a north arrow. Tie bridge dimensions to working points. Show the direction of traffic for each design lane.

##### **Elevation**

Present the primary vertical geometry of the bridge on the elevation view. This consists of vertical curve data, end slopes, existing ground lines, footing elevations, limits of excavation, grading notes, ditch clean out along railroad tracks, and scale. Label bearings as fixed, expansion, or integral. Also label piers, spans, abutments, and slope protection.

For bridges over waterways, provide hydraulic information. Required information includes: channel bottom width, low member elevation, design high water elevation, and assumed flowline elevation.

For grade separation bridges, provide the minimum vertical and horizontal clearances. In addition, provide the dimension from centerline of pier to toe of slope protection. If there is no side pier, give the dimension from toe of slope to centerline of roadway. Dimension the pier, lane, and shoulder widths on the roadway under. Lane slopes on the roadway under are typically omitted, but can be provided if space permits.

When illustrating slope protection use a straight slope line; do not follow the ditch radius curve. To reduce confusion concerning slopes, do not show slopes as 1:2. Many individuals are unsure of whether the first or second number is the horizontal part of the slope. Show the slopes graphically. Where slopes need to be provided in text, explicitly call out the slopes (e.g., 1V: 2H).

### **Typical Cross Section**

The typical cross section is the third general view of the structure. Combined with the general plan and elevation views, the primary geometry of the bridge is conveyed. On the typical cross section show transverse bridge dimensions, lane widths and slopes, beam depth and spacing for all spans, roadway slab and concrete wearing course thicknesses, type of barrier, medians, sidewalks, profile grade location, working line, and all centerlines.

For staged construction projects, provide the in-place, interim, and final cross sections, including temporary anchored or unanchored safety barrier locations.

For complex projects, consider creating a separate plan sheet for pay items and notes for clarity.

### **Utilities**

Show all utilities that may affect bridge construction. Note what is to be done with them (will they be moved, will they no longer be used or do they need to be protected during construction).

### **Miscellaneous**

Provide a Design Data block on the General Plan and Elevation Sheet of the bridge plan set. The information given in the block provides a summary of the primary parameters used for the design. Information in the Design Data block includes: design specifications, design live load, design material properties, future wearing course load assumed in the design, deck area, traffic data, and the operating rating for the new structure. See Appendix 2-C.

Also on this sheet, identify the governing standard specifications for construction. Show a north arrow on the plan view and include a block for engineering certification. Present applicable project numbers on the first sheet; project numbers depend on specific funding sources, so there may be both state and federal project numbers.

Review the title block to ensure it accurately describes the bridge. Within the title block provide span lengths to the nearest foot and the bridge type identification number. The three-character identification number should follow the numbering scheme provided in Appendix 2-A of this manual.

Include any additional standard construction notes and the sheet list for the plan set on the first sheet of the plan set. Provide the schedule of quantities for the entire bridge in tabular form on the second or third sheet of the plan.

Standard practice for placement of bench marks and bridge nameplates is as follows:

- Place a single nameplate on the southeast corner of the bridge for a roadway running north and south. For a roadway running east and west, provide the nameplate at the northeast corner. On twin bridges (two one-directional bridges that are side-by-side, but each have their own bridge number), place a nameplate on each right hand corner approaching bridge. For railroad and pedestrian bridges, place the nameplate on a substructure unit. On bridges that are widened, redecked, or that receive rail modifications that result in additional roadway width, install a new nameplate with the original year completed and the year renovated.
- Do not show a bench mark disk in the bridge plan. Placement of bench mark disks on bridges is no longer needed.

Check if ditch drainage pipe is necessary for the project. If drainage pipe is necessary and the contract has multiple portions (grading, bridge, etc.), identify which portion of the contract contains the pipe. Label ditch drainage pipe on plan and elevation views.

Concrete or aggregate slope protection is used along a highway or railway (grade separation structures). Aggregate slope protection is used more frequently when pedestrian traffic below the bridge is limited. Stream crossings use riprap slope protection supported on a granular or geotextile filter. The Preliminary Bridge Plan will indicate the type of slope protection to be used.



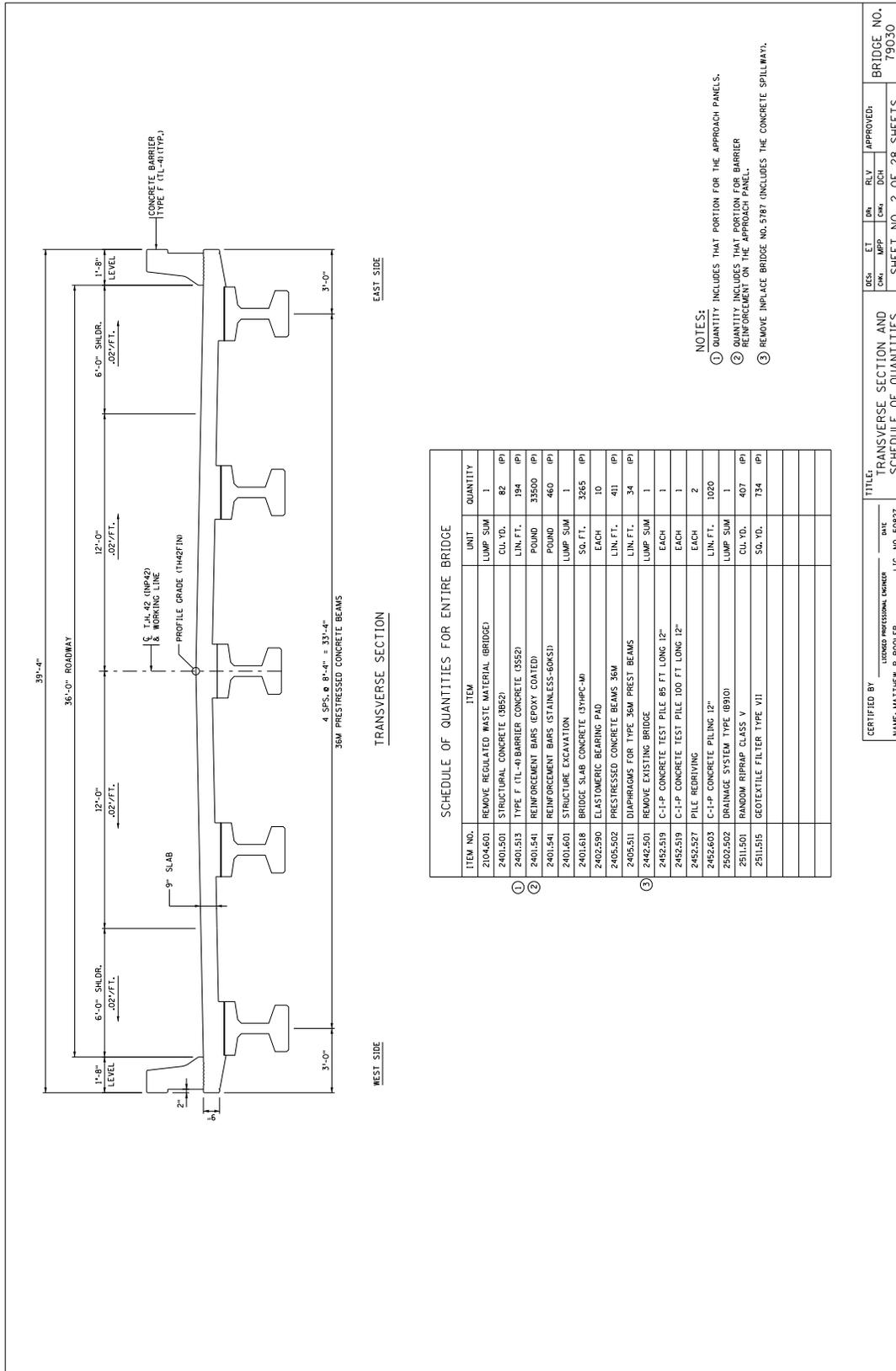


Figure 2.4.2.3.2  
 Typical Cross Section

#### **2.4.2.4 Bridge Layout and Staking Plan**

The Bridge Layout Sheet is used by surveyors to locate the bridge in space with its primary geometry. The primary geometry consists of centerline of roadway(s) and centerline of substructure bearings. Working points are located on substructure bearing centerlines where they are intersected by fascia beam lines and working lines. By providing stationing, X-coordinates, and Y-coordinates for each of the working points, the position of the bridge can be fixed. Figure 2.4.2.4.1 contains an example.

In Figure 2.4.2.4.1, the working line and its azimuth are labeled. Also shown is the angle of intersection between the working line and each of the substructure units and roadways under the bridge. As a primary geometry line, the working line should be labeled throughout the plan set.

Place the control point at the intersection of the survey line and centerline of cross road, track etc. For river crossings, place the control point at an abutment centerline of bearing. Label the control point with its coordinates. Coordinates of the control point and the working points should be given to three decimals of a foot. Tie the working point layout to the control point. Present dimensions in feet (a note on the sheet should say the same).

List the coordinates for all working points in a table labeled "DIMENSIONS BETWEEN WORKING POINTS". Stations and the distances between working points should be presented to the nearest 0.01 foot. Coordinates are assumed to be given in the Minnesota State Plane Coordinate System. If another system is used, place a note on the sheet identifying the system used.

In addition to horizontal geometry, a limited amount of vertical geometry is provided on the Bridge Layout Sheet. The vertical geometry consists of elevations and drops. The elevation at the top of roadway and the bridge seat is provided for all working points located on beam lines and is appended to the "DIMENSIONS BETWEEN WORKING POINTS" table.

Drop or elevation difference information is provided for each substructure unit. Drop information is summarized in the "TOP OF ROADWAY TO BRIDGE SEAT" table. The table should contain the following items:

- 1) Deck Thickness
- 2) Stool Height
- 3) Beam Height
- 4) Bearing Height
- 5) Total Height

If the drop dimension is the same for all beam lines, provide a single value for each substructure unit. If the drop dimensions vary at substructure locations, provide a value for each beam line. Total values should be given in both inches and decimals of a foot to two places.



#### **2.4.2.5 Standard Abbreviations**

Use standard abbreviations to clarify information on plan sets and reduce the clutter on a crowded plan sheet. Appendix 2-B presents a list of standard abbreviations that can be utilized in a plan. Define abbreviations used in a plan set on the sheet where they are used or as part of a General Notes sheet.

#### **2.4.2.6 Inclusion of Standard Bridge Details and Bridge Standard Plans in Plan Sets**

There are two parts to the Standard Bridge Details Manual: Part I and Part II. They are published on the Bridge Office Web site at:  
<http://www.dot.state.mn.us/bridge/standards.html>

Bridge details are intended, where applicable, to be incorporated into a set of bridge plans.

Bridge Details Manual Part I, typically referred to as the B-Details, are presented in a "portrait" orientation on an 8 1/2" x 11" sheet. Included are details for nameplates, pile splices, bearings, diaphragms, steel superstructures, floor drains, and other miscellaneous details.

Bridge Details Manual Part II consists of details that occupy an entire plan sheet. The majority of these details are for barriers, parapets, medians, prestressed concrete beams, and expansion joints.

Similar to Bridge Details Manual Part II, details from the Bridge Standard Plans Manual are intended to be incorporated into bridge plan sets and occupy an entire plan sheet. The information presented may be much more in-depth as the information for multiple designs is presented on a single sheet. Bridge Standard Plans are only available for precast concrete box culverts.

How standard details are incorporated into a bridge plan will depend on the amount of revisions needed to the details, falling into 3 categories:

- 1) Standard is unmodified: This is when the standard is used as drawn with no changes, or with all blanks filled in where expected. Box out and place cross-hatching across all specific details that do not apply. Do not remove them. It is not necessary to cross out alternative sizes in tables or alternate dimensions that are not used, or circled notes that are only referred to in the details that do not apply.
- 2) Standard is modified: This is when details, dimensions, or notes on the standard have been specifically modified from what is shown in the standard. In this case, place the word "MODIFIED" under the B-Detail or after the Figure Number. Also add a box containing

a note stating what was modified to help plan readers quickly locate them. Do not “cloud” the changes.

- 3) Standard is substantially modified: This is when much of the standard is changed by extensive and/or numerous modifications to details, dimensions, and notes to the extent that it ceases to reflect the standard. It will require a judgment call on the part of the engineer as to when this category applies. In this case, remove the State Bridge Engineer approval block and the figure number. Note that for this case, the engineer takes full responsibility for the details shown on the sheet.

Note that these rules do not apply to box culverts, which are included with the road plan. Modification of box culvert standards is to be done in accordance with the Design Scene, Chapter 3 requirements, which can be found at:

<http://www.dot.state.mn.us/pre-letting/scene/index.html>

#### **2.4.2.7 Standard Plan Notes**

Similar to other plan elements, standard plan notes have been prepared to increase the consistency of information presented on final design plans. Plan notes serve a variety of purposes; they communicate design criteria, specific construction requirements, and a variety of notes pertaining to the construction or fabrication of specific bridge elements. Appendix 2-C contains the Standard Plan Notes. These notes have been grouped into the following categories:

- Design Data and Projected Traffic Volumes
- Construction Notes
- Signature Block
- Drainage and Erosion Control
- Excavation and Earthwork
- Reinforcement
- Foundations
- Steel Materials, Fabrication, and Erection
- Concrete Placements
- Welded Steel Bearing Assemblies
- Cutting and Removal of Old Concrete
- Joints and Joint Sealer
- Timber Bridges
- Miscellaneous

Designers unfamiliar with MnDOT’s Standard Plan Notes should review the list prior to beginning final design. Reviewing the notes prior to design will familiarize designers with the material properties to be used, and other constraints typically placed on construction. Perform a second review of

the notes at the end of design to ensure that all applicable notes were incorporated into the plan set.

#### **2.4.2.8 Quantity Notes and Pay Items**

##### **Standard Summary of Quantities Notes**

During construction, contractors are compensated according to the work they complete. The value of the work item is identified when the contractor submits their bid. For each work item or pay item the contractor must supply a price. The pay items are coordinated with specifications and special provisions. To clarify what is included in a specific pay item, the Bridge Office has assembled a Standard Summary of Quantity Notes. Like other plan elements, these notes help ensure uniformity across plan sets and permit MnDOT to generate a historical price database that can be used to estimate the cost of future bridges. The Standard Summary of Quantities Notes for bridge projects is listed in Appendix 2-D.

##### **Pay Items**

The current MnDOT list of pay items (commonly referred to as the Trns\*port List) is located at the following link:

<http://transport.dot.state.mn.us/Reference/refItem.aspx>

When populating the "SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE" in the bridge plan, it is important to note the following:

- 1) The Trns\*port List shows a 12 digit number under the "ITEM NUMBER" column. Provide only the first seven numbers including the decimal point (all the numbers before the /) for each pay item in the bridge plan.
- 2) The Trns\*port List shows two descriptions for each pay item. Choose the "ITEM SHORT DESCRIPTION" column for all pay item descriptions in the bridge plan.
- 3) Use the "PLAN UNIT NAME" column from the Trns\*port List for listing the units in the bridge plan.
- 4) The item number, item description, and units must read exactly as shown in the Trns\*port List, including spaces, dashes, parentheses, etc. In order to avoid errors, it is suggested to copy and paste them directly from the list into the bridge plan.

For each pay item shown in the plan, provide a reconciled quantity estimate. Some pay items are to be designated as "plan quantity pay items", for which payment to the contractor will be based on the quantity given in the plan rather than measuring in the field. For these items, include a "(P)" as an appendix to the item label. For example:

2401.507 STRUCTURAL CONCRETE (3B52) 699 CU YD (P)

Pay items are to be designated as “plan quantity pay items” when the quantity for payment can be calculated using the dimensions given in the plan and the dimensions are not expected to change in the field. Some examples include:

- structural concrete paid for by the cubic yard
- bridge slab concrete paid for by the square foot
- reinforcement bars paid for by the pound
- prestressed concrete beams paid for by the linear foot
- ornamental metal railing paid for by the linear foot

Some examples of pay items that are **not** “plan quantity pay items” include:

- piling paid for by the linear foot (because plan quantity is an estimate only and final pile lengths are determined in the field)
- random riprap paid for by the cubic yard (because plan dimensions are approximate and actual volume may differ)
- removal and patching of concrete slabs paid for by the square foot (because actual area will be determined in the field)
- structural excavation paid for by lump sum (because it is not a calculated value based on plan dimensions)
- bearing assemblies paid for by the each (because it is not calculated using dimensions from the plan)

### Miscellaneous

Round off quantities to the nearest pay item unit except for the following:

- Earth excavation to nearest 10 cubic yards.
- Reinforcement bars and structural steel to nearest 10 pounds.
- Piling lengths to nearest 5 feet.

When computing small bituminous quantities use the following:

Wearing course = 110 pounds / square yard / inch thickness

Shoulder or Wearing Course (6.5%)

0.065 (thickness in inches) (110 pounds) = \_\_\_ pounds / sq. yard

Tack Coat = 0.03 gallons / square yard

Binder or Base Course (5.3%)

0.053 (thickness in inches) (110 pounds) = \_\_\_\_ pounds / sq. yard

Compute deck area (rounded to the nearest square foot) by multiplying the transverse out-to-out bridge width by the longitudinal end-of-deck to end-of-deck distance. (Do not include bridge approach panels or paving brackets.)

When computing structural steel quantities, increase the calculated weight by 1.5% to account for the weight of steel for welds and bolt stick-through.

### **2.4.3 Revised Sheets**

Sometimes, revisions to the plan are required after the letting due to an error found in the plan or other issues that arise during construction. When this occurs, use the following procedure:

- 1) Revise the sheet as follows (See Figure 2.4.3.1):
  - i. Make the necessary revisions to the sheet.
  - ii. In the revision block, provide the revision number within a triangle border, the revision date, a description of the revision, and the initials of the engineer who approved the revision.
  - iii. "Cloud" the actual revisions to the sheet and include the revision number within a triangle border next to the "clouded" change. When sheets have been revised multiple times, remove previous revision "clouds", only "clouding" the current revisions. However, leave previous triangles with their revision numbers in place.
  - iv. Change the sheet number by placing a "-R" and the revision number after the original sheet number. For example, revision 1 to sheet 7 will be designated "SHEET NO. 7-R1", revision 2 will be designated "SHEET NO. 7-R2", etc. For situations where an additional plan sheet must be inserted as part of the revision, repeat the preceding sheet number with an "A" after it. For example, as part of revision 1 where a sheet needs to be added between sheet 5 and 6, designate the revised sheet as "SHEET NO. 5A-R1".
- 2) Plot and certify the revised sheet.
- 3) Draft a transmittal letter from the State Bridge Design Engineer to the Resident Engineer in the District construction office. Submit the letter and the revised sheet to the State Bridge Design Engineer for signature and distribution. Memo templates are available on the Bridge Office network drive. Consultants should contact MnDOT Bridge Office Project Manager to obtain the file.

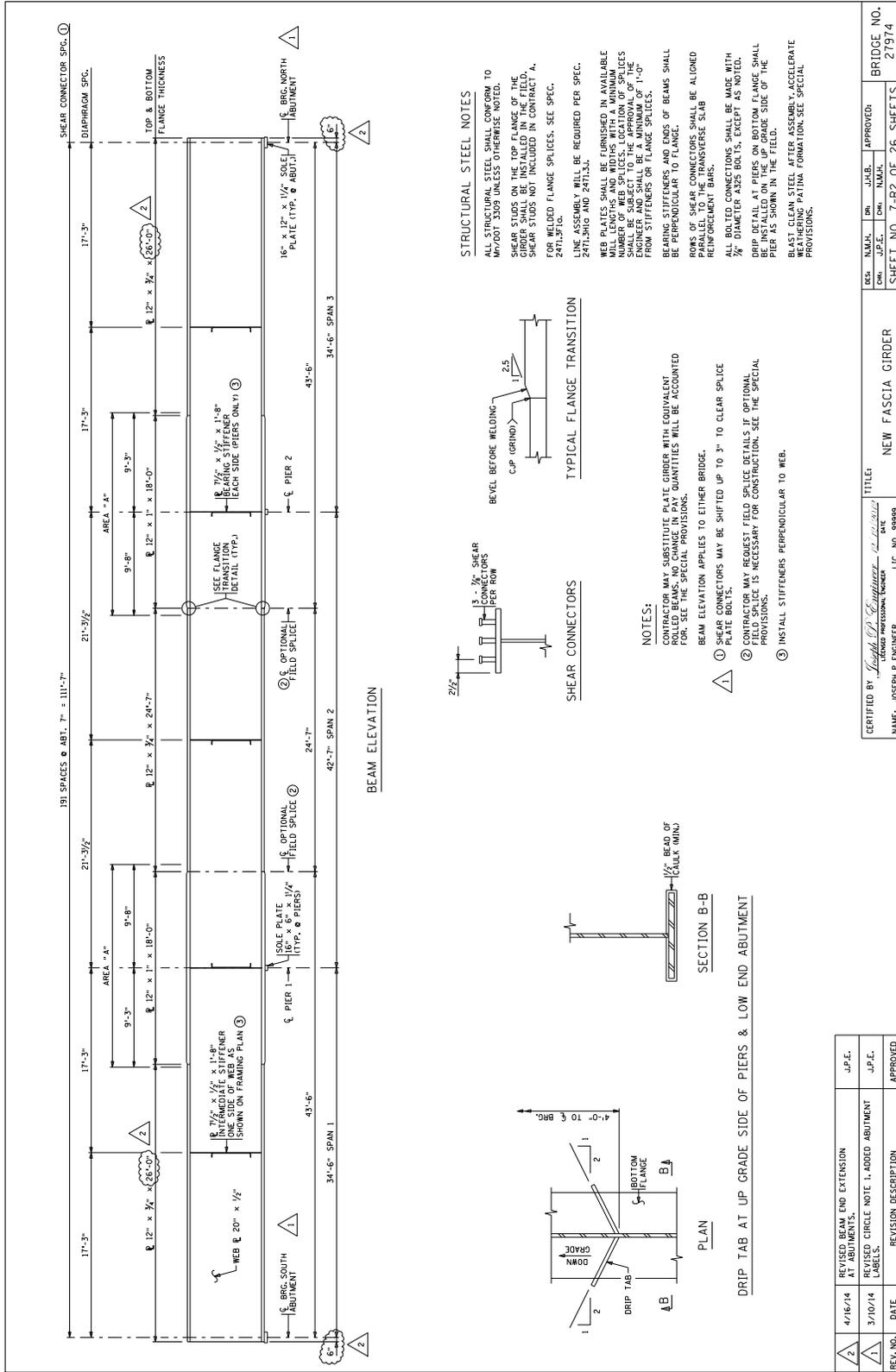
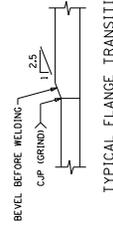


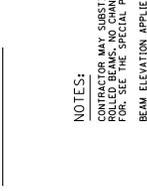
Figure 2.4.3.1

**STRUCTURAL STEEL NOTES**

ALL STRUCTURAL STEEL SHALL CONFORM TO A572/508 UNLESS OTHERWISE NOTED.  
 SHEAR STUDS ON THE TOP FLANGE OF THE GIRDER SHALL BE INSTALLED IN THE FIELD. SHEAR STUDS NOT INCLUDED IN CONTRACT A, FOR WELDED FLANGE SPLICES. SEE SPEC. 2411.5F10.  
 LINE ASSEMBLY WILL BE REQUIRED PER SPEC. 2411.5H10 AND 2411.5J.  
 WEB PLATES SHALL BE FURNISHED IN AVAILABLE LENGTHS. THE NUMBER OF SPLICES AND NUMBER OF WEB SPLICES, LOCATION OF SPLICES SHALL BE SUBJECT TO THE APPROVAL OF THE CONTRACTOR. THE DISTANCE FROM THE CENTERLINE OF THE GIRDER TO THE CENTERLINE OF THE SPLICES SHALL BE PER STIFFENERS OR FLANGE SPLICES. 1'-0" BEARING STIFFENERS AND ENDS OF BEAMS SHALL BE PERPENDICULAR TO FLANGE.  
 ROWS OF SHEAR CONNECTORS SHALL BE ALIGNED PARALLEL TO THE TRANSVERSE SLAB REINFORCEMENT BARS.  
 ALL BOLTED CONNECTIONS SHALL BE MADE WITH 3/4" DIAMETER A505 BOLTS, EXCEPT AS NOTED. DRIP DETAIL AT PIERS ON BOTTOM FLANGE SHALL BE PER DETAIL AT UP GRADE SIDE OF THE PIER AS SHOWN IN THE FIELD.  
 BLAST CLEAN STEEL AFTER ASSEMBLY ACCELERATE WEATHERING PATINA FORMATION. SEE SPECIAL PROVISIONS.



**SHEAR CONNECTORS**



**NOTES:**

- CONTRACTOR MAY SUBSTITUTE PLATE GIRDER WITH EQUIVALENT ROLLED BEAMS. NO CHANGE IN PAY QUANTITIES WILL BE ACCOUNTED FOR. SEE THE SPECIAL PROVISIONS.
- BEAM ELEVATION APPLIES TO EITHER BRIDGE.
- SHEAR CONNECTORS MAY BE SHIFTED UP TO 3" TO CLEAR SPLICE PLATE BOLTS.
- CONTRACTOR MAY REQUEST FIELD SPLICE DETAILS IF OPTIONAL FIELD SPLICE IS NECESSARY FOR CONSTRUCTION. SEE THE SPECIAL PROVISIONS.
- INSTALL STIFFENERS PERPENDICULAR TO WEB.

DES. CHG.	NAME	DR.	DATE	APPROVED:	BRIDGE NO.
					21914
CERTIFIED BY: Joseph P. Engstrom, Licensed Professional Engineer NAME: JOSEPH P. ENGINEER LIC. NO. 99999					SHEET NO. 7-R2 OF 26 SHEETS TITLE: NEW FASCIA GIRDER APPROVED:

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**2.5 Reconstruction Guidelines and Details** Typical details for the reconstruction of barriers, superstructure joints, and pavement joints are presented in this section.

**2.5.1 Superstructure** [Future manual content]

**2.5.1.1 Barriers** [Future manual content]

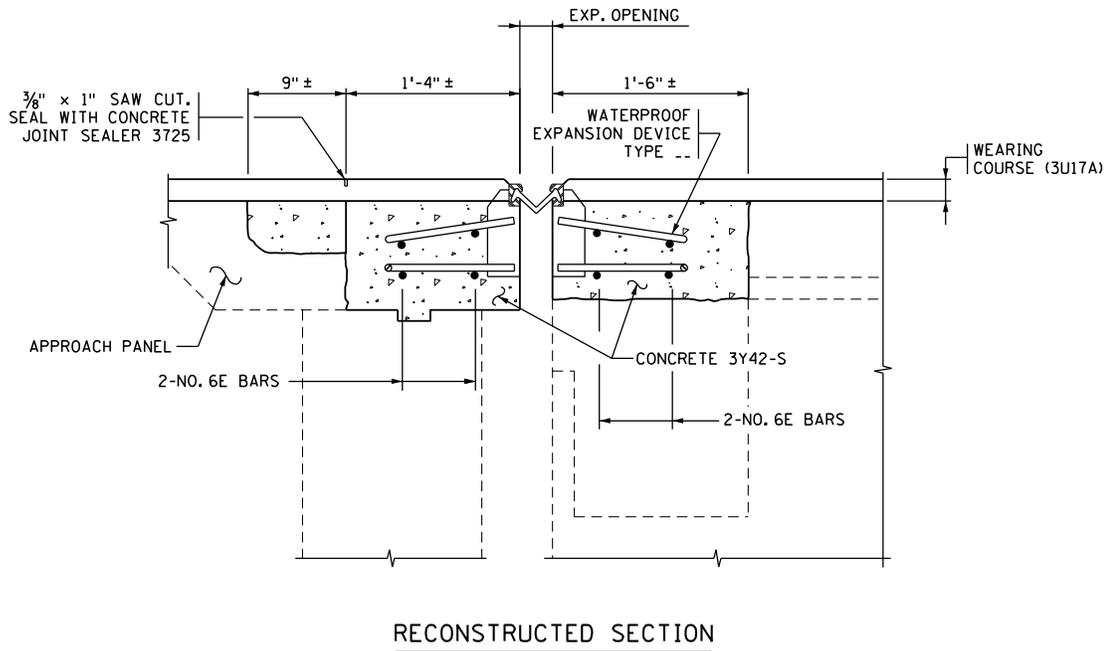
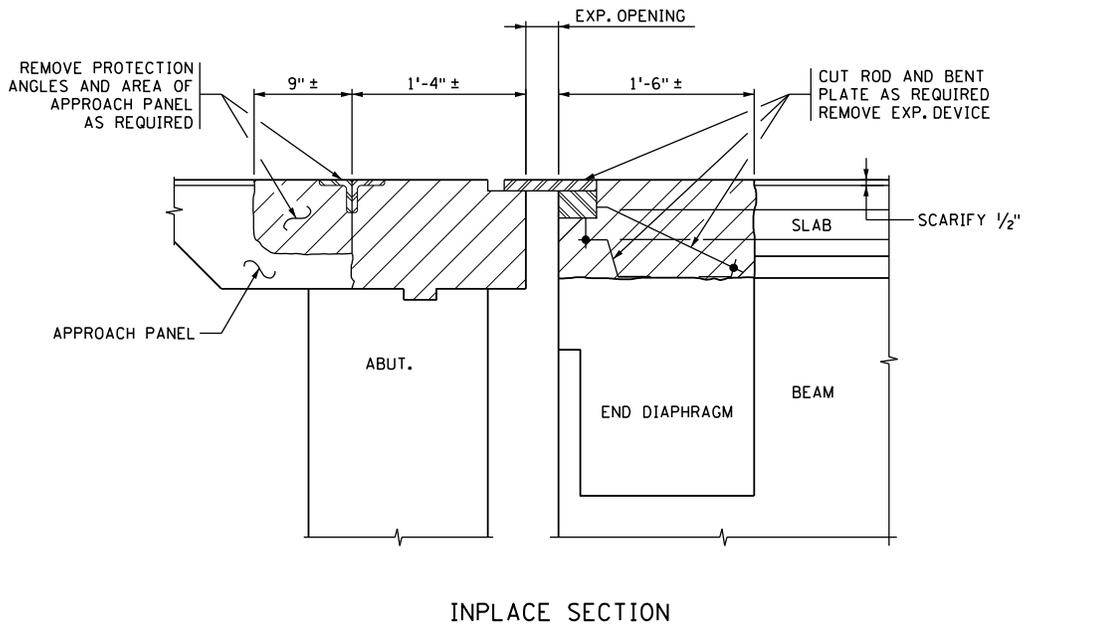
**2.5.1.2 Wearing Course** [Future manual content]

**2.5.1.3 Expansion/Fixed Joints** The following figures show typical details for the reconstruction of expansion joints and fixed joints:

- Figure 2.5.1.3.1 Reconstruct Expansion Joint Type A
- Figure 2.5.1.3.2 Reconstruct Expansion Joint Type B
- Figure 2.5.1.3.3 Reconstruct Expansion Joint Type C
- Figure 2.5.1.3.4 Reconstruct Expansion Joint Type D
- Figure 2.5.1.3.5 Reconstruct Expansion Joint Type D
- Figure 2.5.1.3.6 Reconstruct Expansion Joint Type E
- Figure 2.5.1.3.7 Reconstruct Expansion Joint Type F
- Figure 2.5.1.3.8 Reconstruct Fixed Joint Type A
- Figure 2.5.1.3.9 Reconstruct Fixed Joint Type A

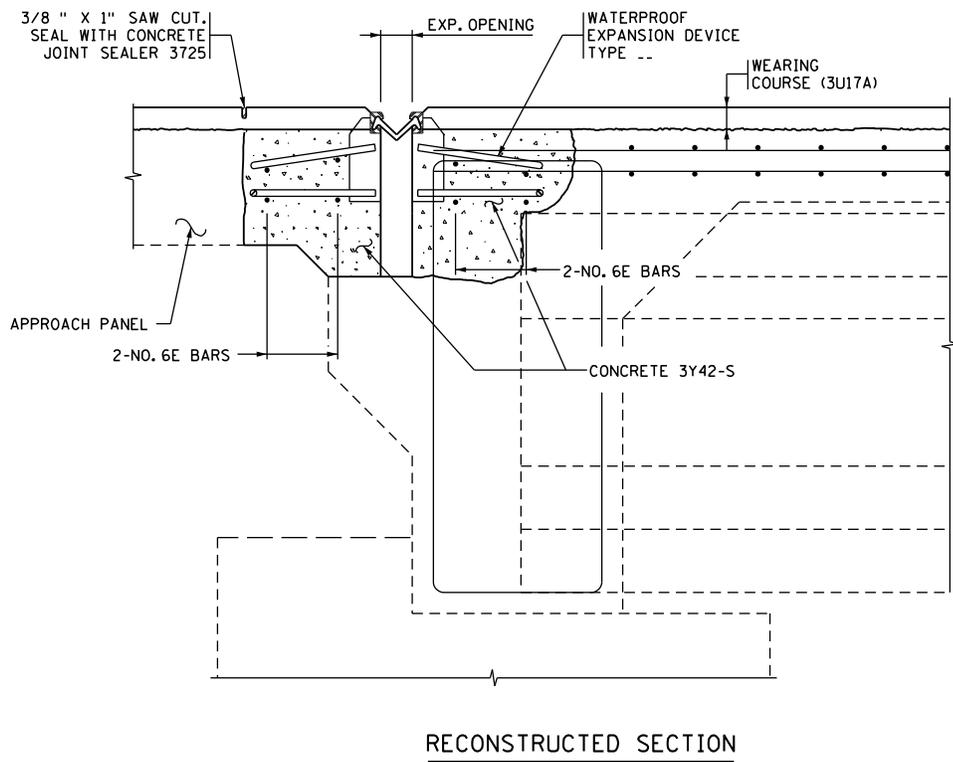
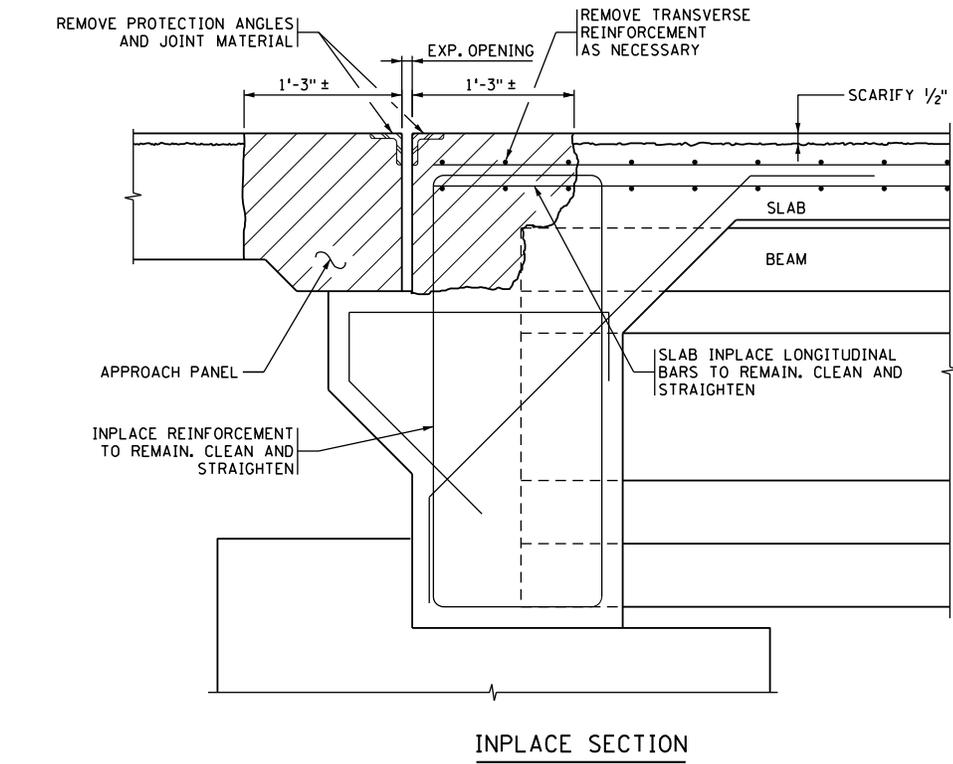
**Expansion/Fixed Joint Reconstruction Pay Items**

- Item No. 2433.603 "Reconstruct Expansion Joint, Type \_\_\_\_\_", Lin. Ft.
  - Type A – Replace sliding plate or in-place waterproof device with new waterproof joint.
  - Type B – Slab over parapet and contraction type abutments replace joint with waterproof joint at same location.
  - Type C – Slab over parapet, replace joint with waterproof joint at front of parapet.
  - Type D – Replace cork joint at pier with waterproof joint.
  - Type E – Replace joint at hinge with waterproof joint.
  - Type F – Replace finger joints with waterproof joint or raise device and place a waterproof trough.
  - Type Special – None of the above or a combination of the above.
  
- Item No. 2433.603 "Reconstruct Fixed Joint, Type \_\_\_\_\_", Lin. Ft.
  - Type A – Eliminate in-place joint
  - Type B – Install waterstop
  - Type Special – None of the above



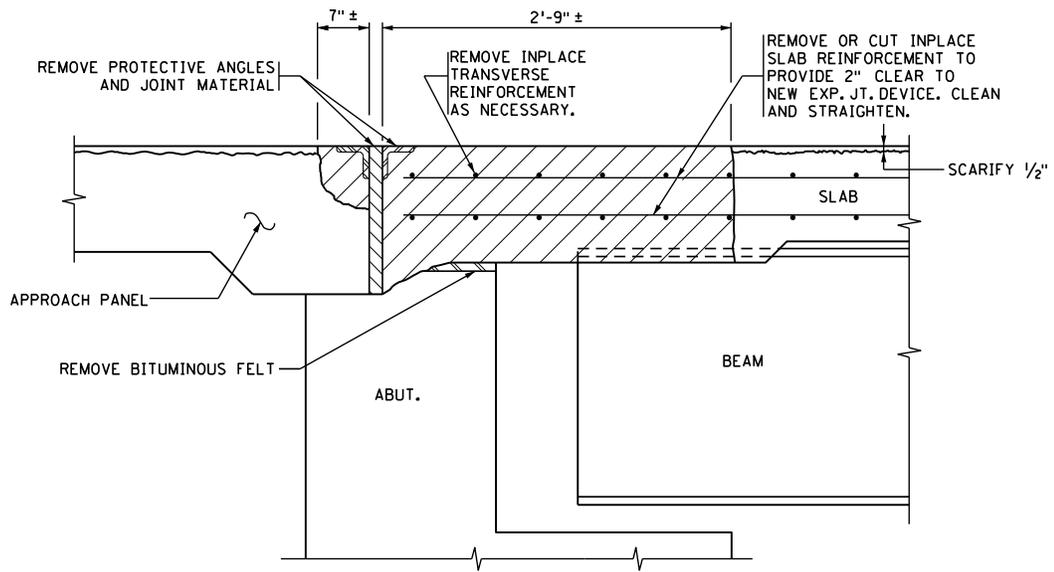
RECONSTRUCT EXPANSION JOINT TYPE A

**Figure 2.5.1.3.1**  
**Expansion Joints**

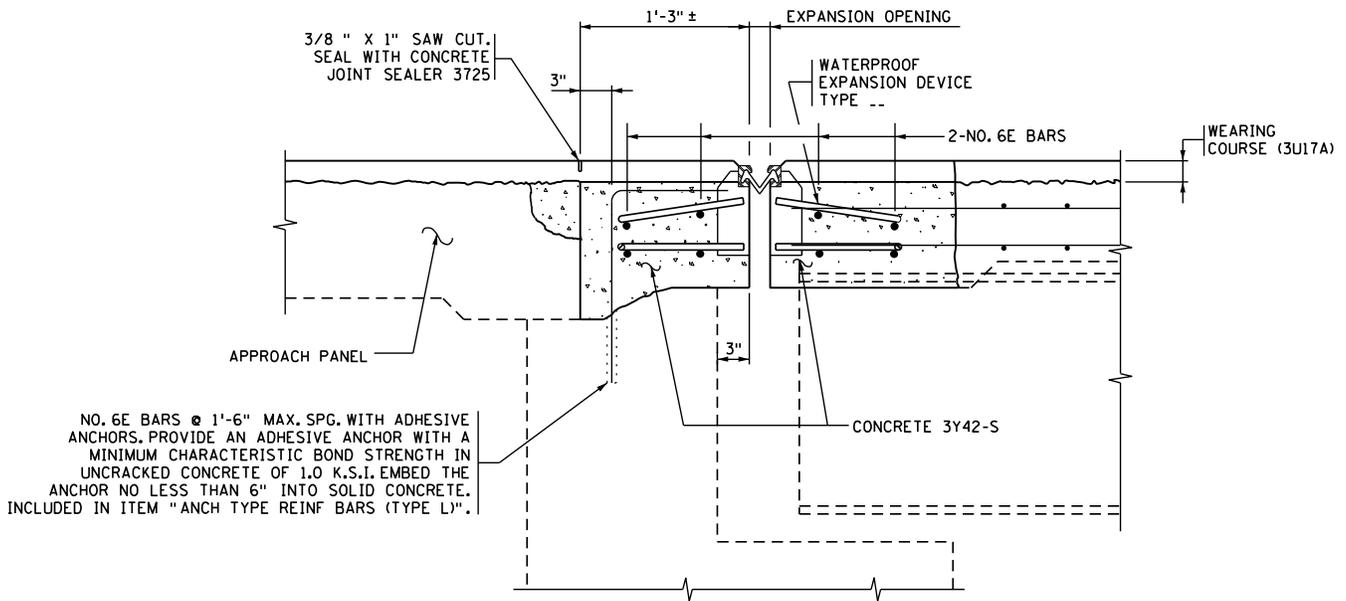


**RECONSTRUCT EXPANSION JOINT TYPE B**

**Figure 2.5.1.3.2**  
**Expansion Joints**



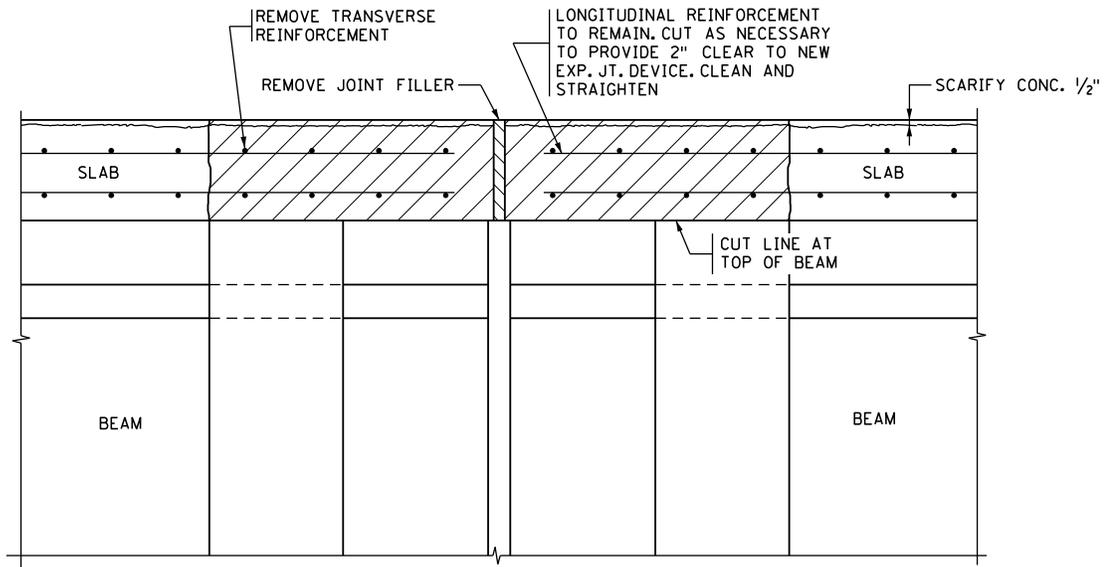
INPLACE SECTION



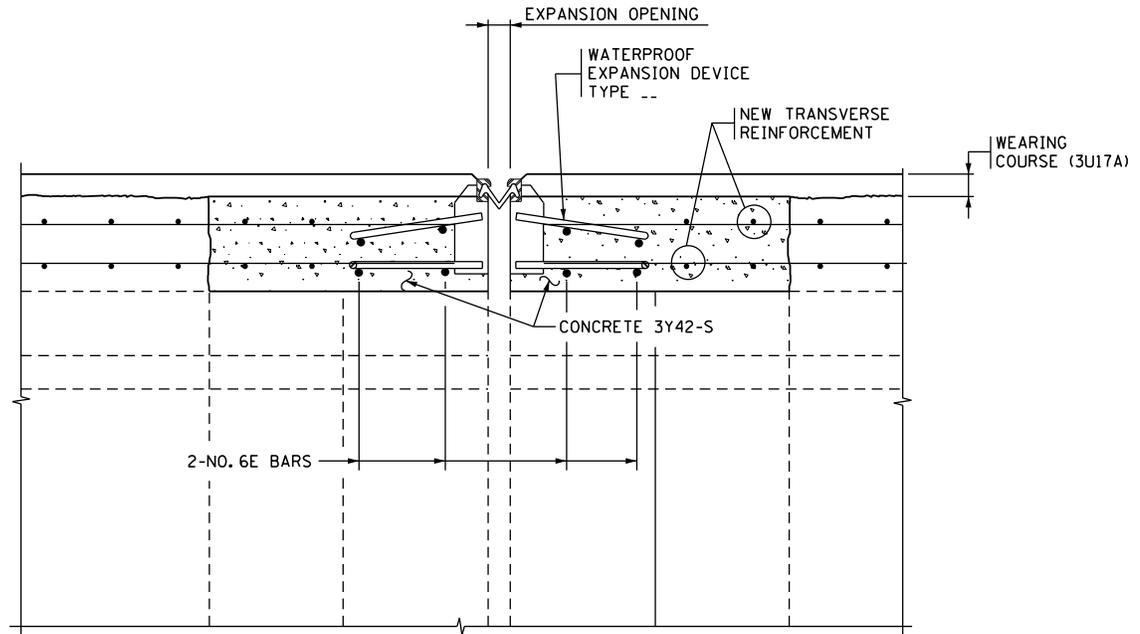
RECONSTRUCTED SECTION

RECONSTRUCT EXPANSION JOINT TYPE C

**Figure 2.5.1.3.3**  
**Expansion Joints**



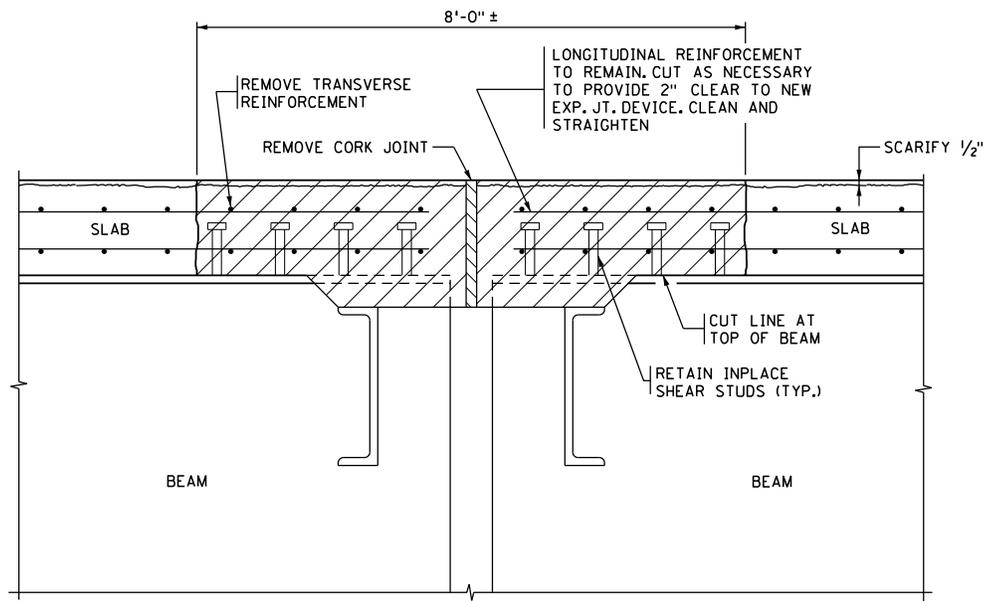
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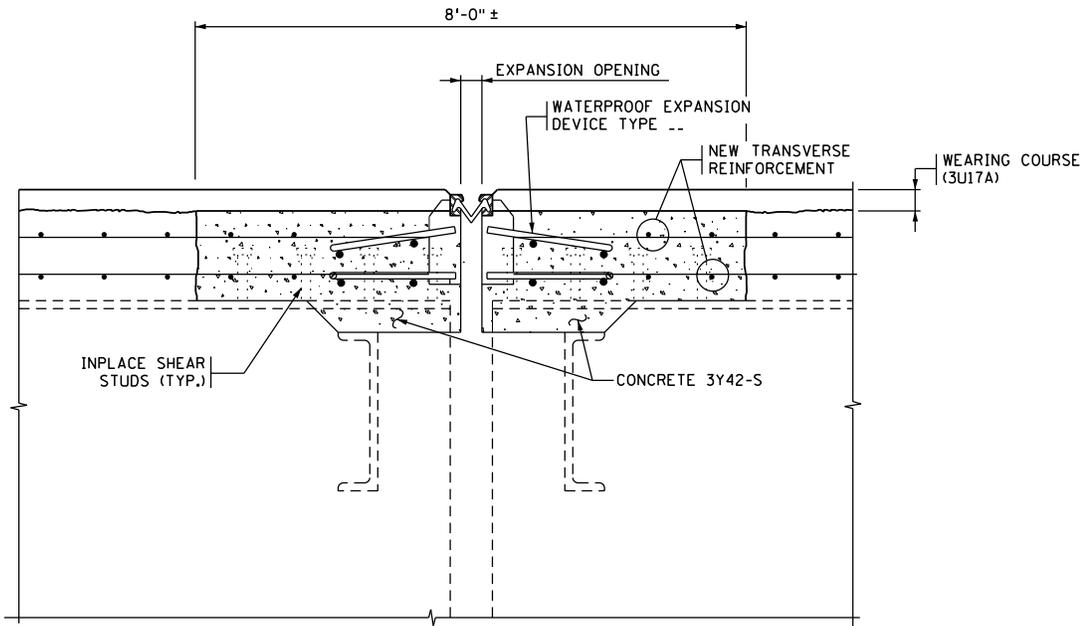
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RECONSTRUCT EXPANSION JOINT TYPE D

**Figure 2.5.1.3.4**  
**Expansion Joints**



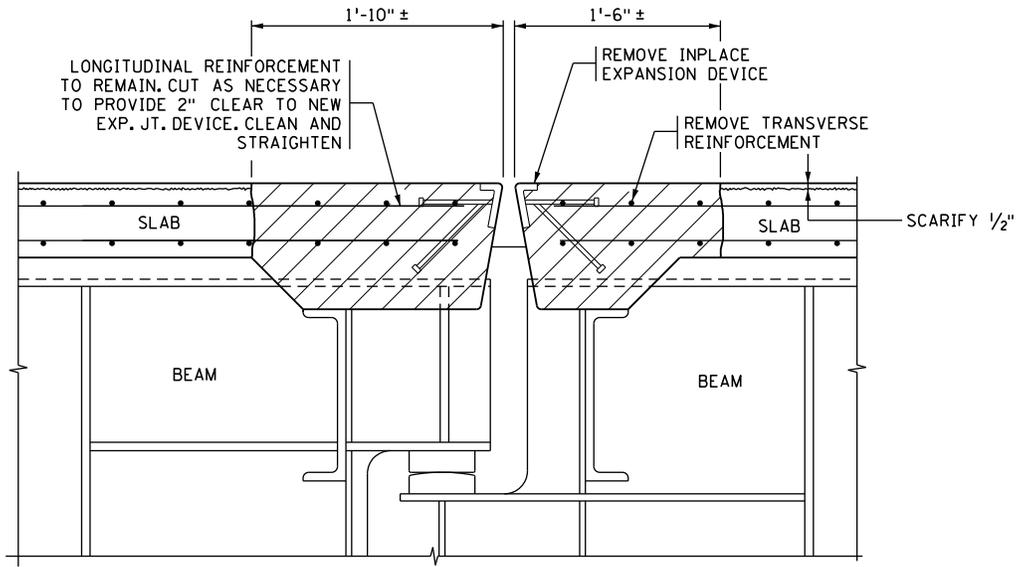
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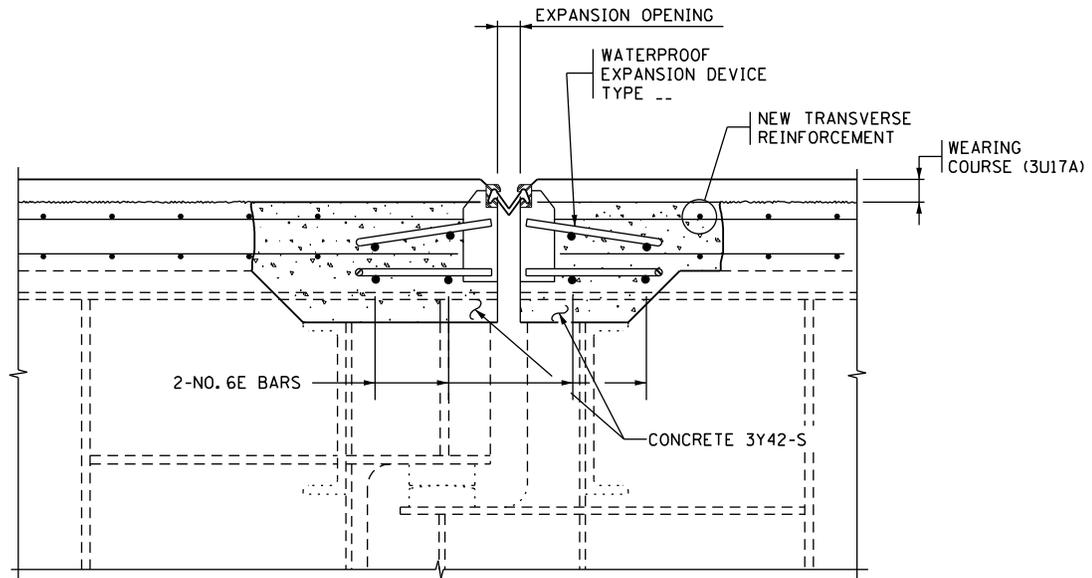
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RECONSTRUCT EXPANSION JOINT TYPE D

**Figure 2.5.1.3.5**  
**Expansion Joints**



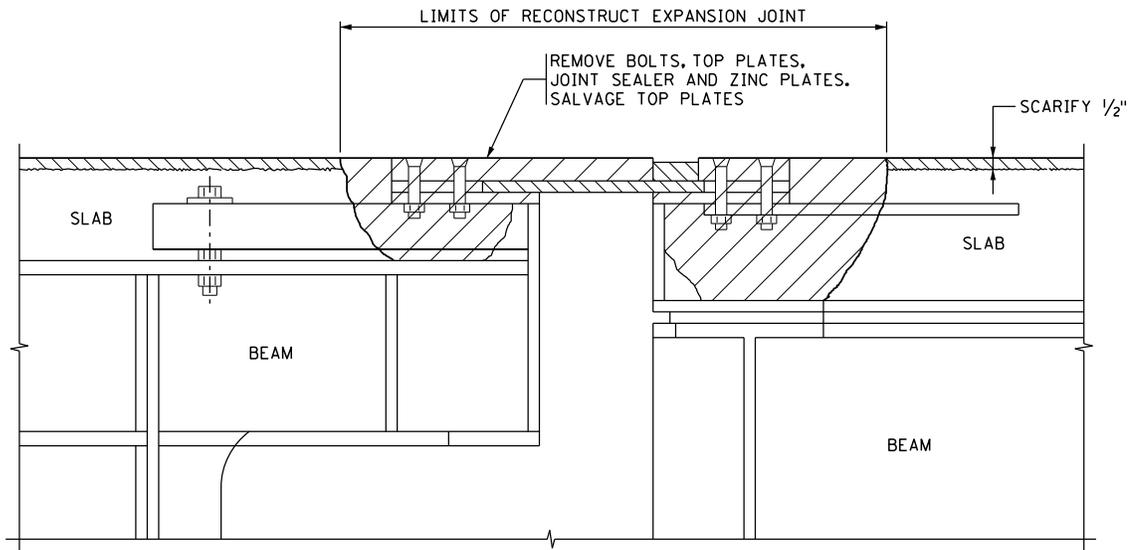
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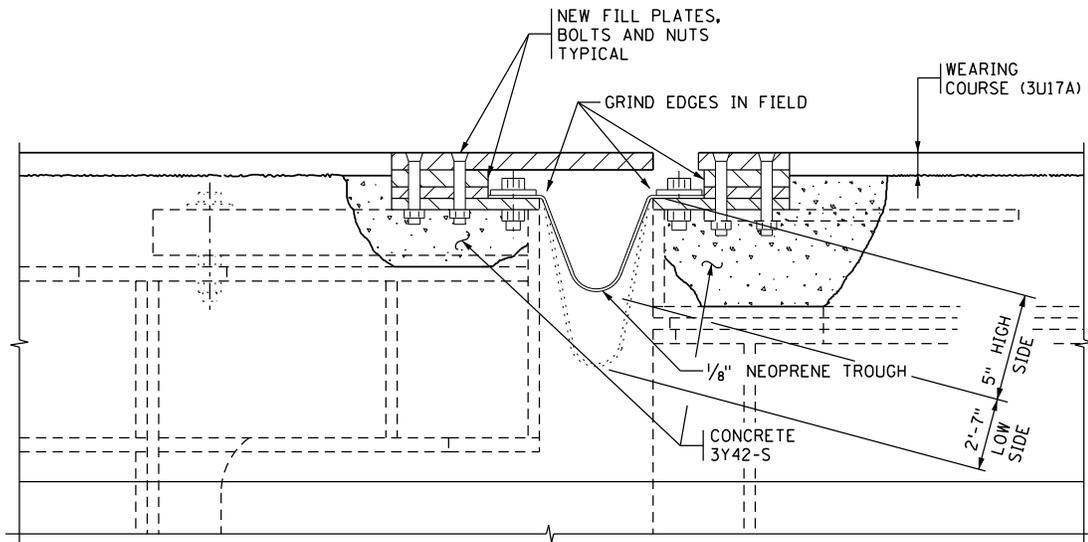
RECONSTRUCTED SECTION

RECONSTRUCT EXPANSION JOINT TYPE E

**Figure 2.5.1.3.6**  
**Expansion Joints**



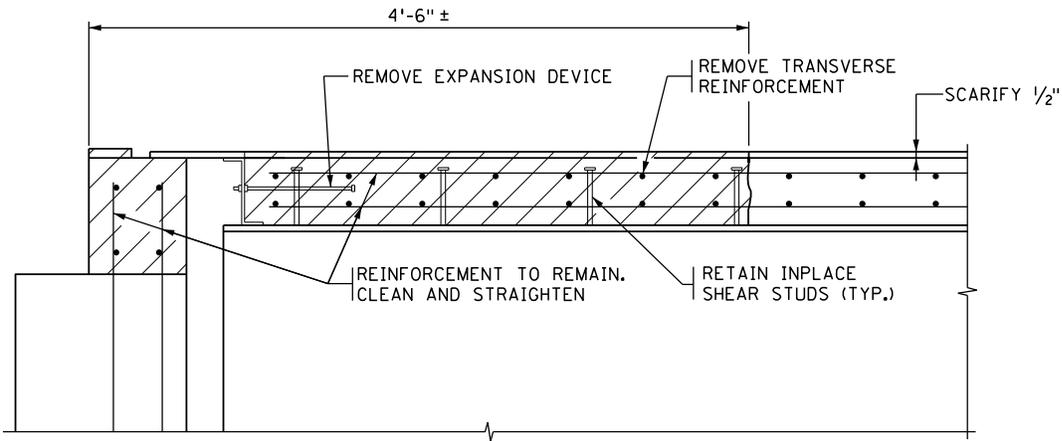
INPLACE SECTION



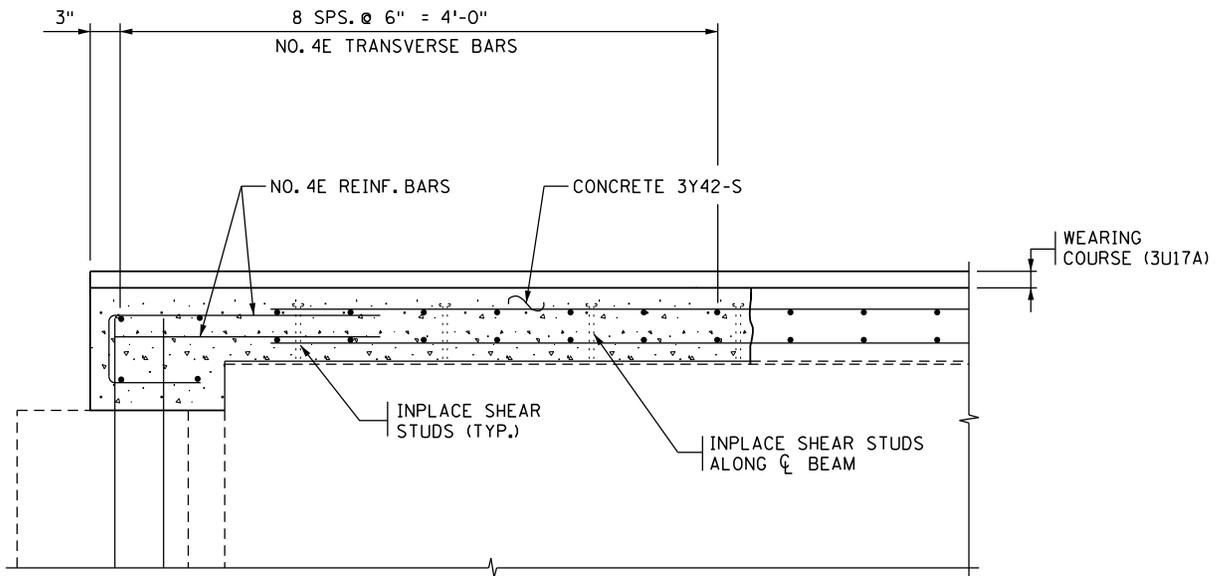
RECONSTRUCTED SECTION

RECONSTRUCT EXPANSION JOINT TYPE F

**Figure 2.5.1.3.7**  
**Expansion Joints**



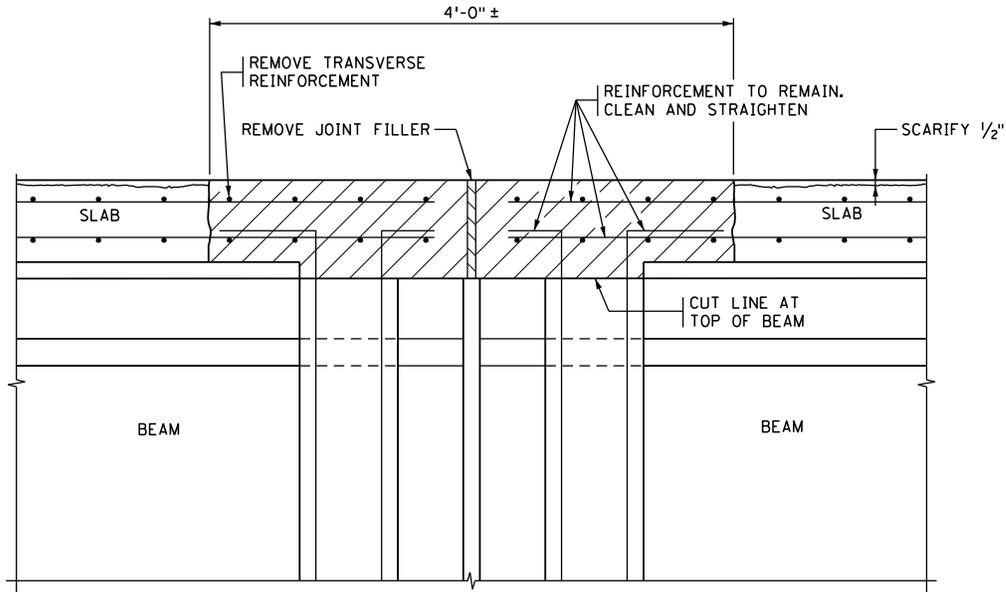
INPLACE SECTION AT ABUTMENTS



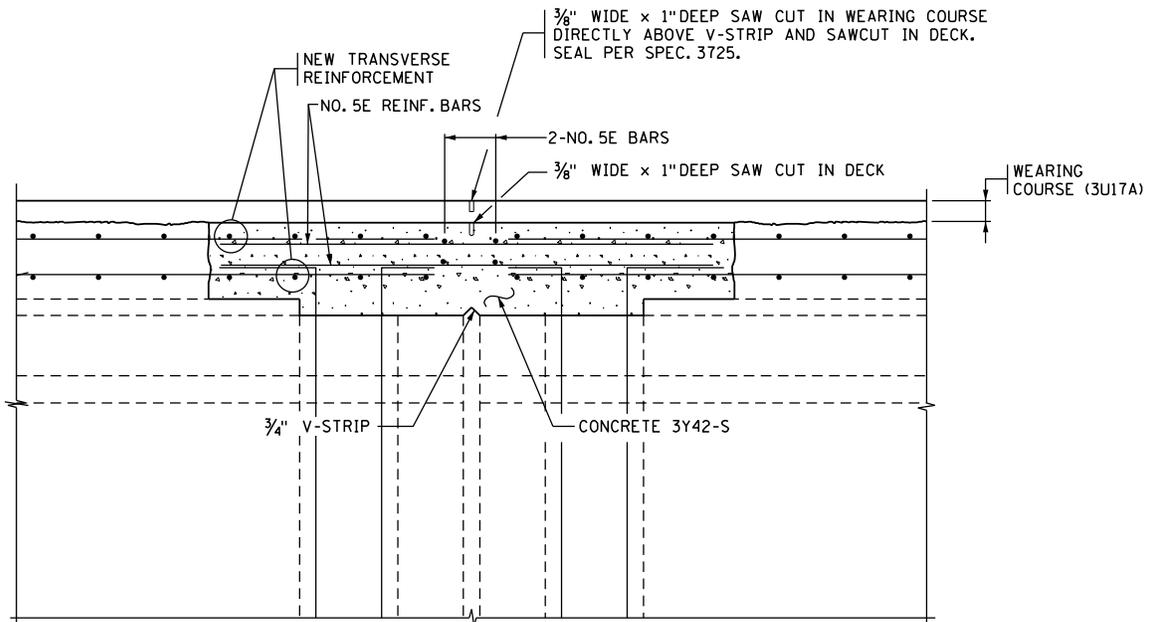
RECONSTRUCTED SECTION AT ABUTMENTS

RECONSTRUCT FIXED JOINT TYPE A

**Figure 2.5.1.3.8**  
**Fixed Joints**



INPLACE SECTION OVER PIERS



RECONSTRUCTED SECTION OVER PIERS

RECONSTRUCT FIXED JOINT TYPE A

**Figure 2.5.1.3.9**  
**Fixed Joints**

**2.5.2 Substructure** [Future manual content]

**2.5.2.1 Abutments** The following figures show typical details for the repair of abutment paving brackets:

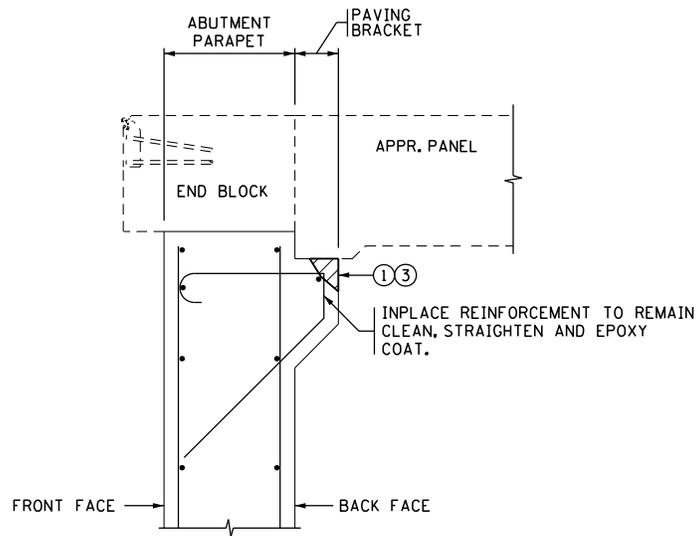
Figure 2.5.2.1.1 Repair Paving Bracket

Figure 2.5.2.1.2 Reconstruct Paving Bracket

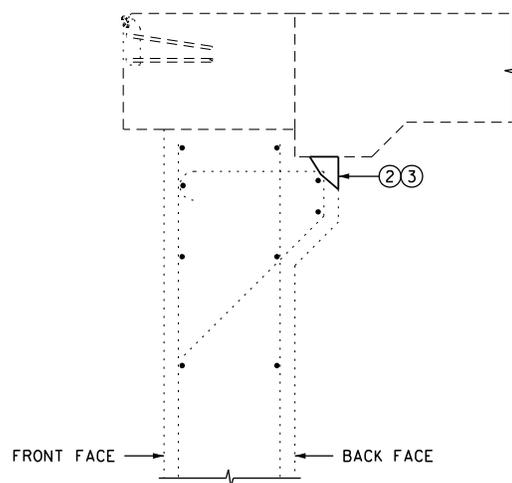
Figure 2.5.2.1.3 Reconstruct Paving Bracket and Wall

**Paving Bracket Repair Pay Items**

- Item No. 2433.603 "Repair Paving Bracket", Lin. Ft.
- Item No. 2433.603 "Reconstruct Paving Bracket", Lin. Ft.
- Item No. 2433.618 "Reconstruct Paving Bracket and Wall", Sq. Ft.



INPLACE SECTION REMOVAL

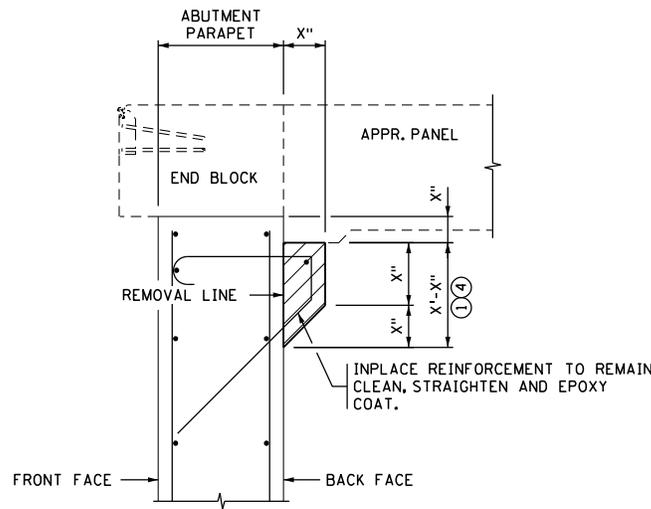


REPAIRED SECTION

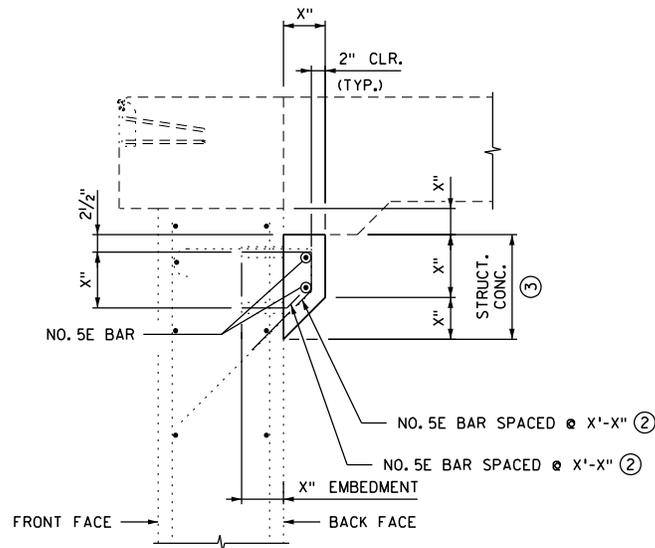
- ① REMOVE ANY DETERIORATED AREAS OF PAVING BRACKET TO SOUND CONCRETE IN ACCORDANCE WITH THE SPECIAL PROVISIONS.
- ② REPLACE REMOVED CONCRETE WITH CONCRETE MIX NO. 3B52. COAT SURFACE OF EXISTING CONCRETE WITH BONDING GROUT PER SPECIAL PROVISIONS PRIOR TO CASTING.
- ③ INCLUDED IN ITEM "REPAIR PAVING BRACKET".

REPAIR PAVING BRACKET

**Figure 2.5.2.1.1**  
**Paving Brackets**



INPLACE SECTION REMOVAL

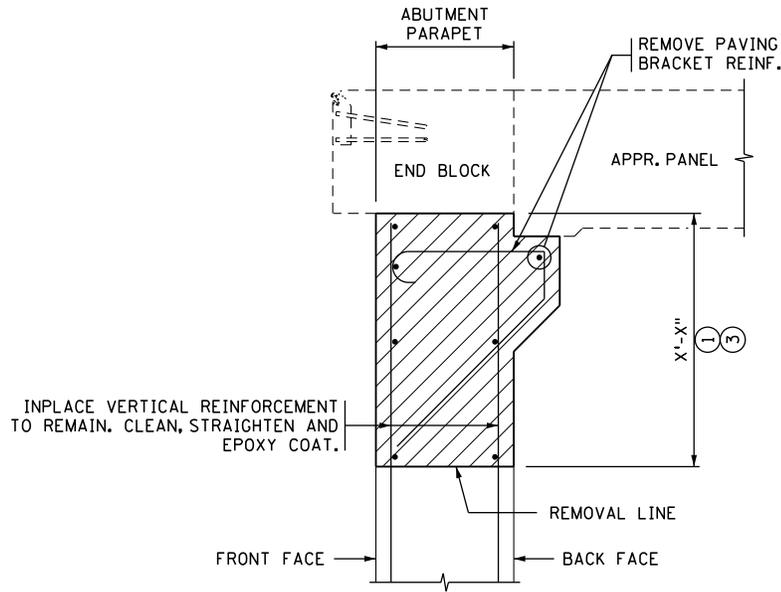


RECONSTRUCTED SECTION ④

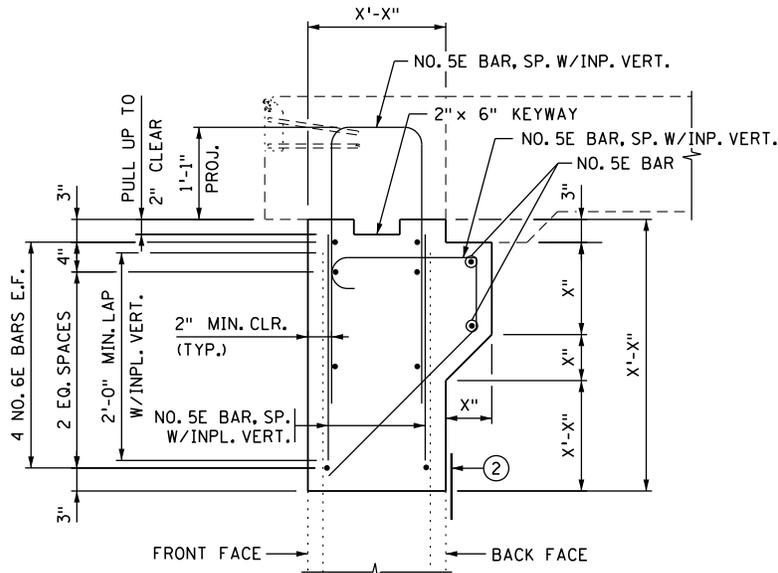
- ① REMOVE PAVING BRACKET IN ITS ENTIRETY TO SOUND CONCRETE IN ACCORDANCE WITH THE SPECIAL PROVISIONS.
- ② SUPPLEMENT INPLACE REINFORCEMENT AS NEEDED WITH NEW BARS AND ADHESIVE ANCHORS. PROVIDE AN ADHESIVE ANCHOR WITH A MINIMUM CHARACTERISTIC BOND STRENGTH IN UNCRACKED CONCRETE OF 1.0 K.S.I. EMBED THE ANCHORAGE NO LESS THAN 6" INTO SOLID CONCRETE. INCLUDED IN ITEM "ANCH TYPE REINF BARS (TYPE L)".
- ③ REPLACE CONCRETE WITH CONCRETE MIX NO. 3B52. COAT SURFACE OF EXISTING CONCRETE WITH BONDING GROUT PER SPECIAL PROVISIONS PRIOR TO CASTING.
- ④ INCLUDED IN ITEM "RECONSTRUCT PAVING BRACKET" EXCEPT AS NOTED.

RECONSTRUCT PAVING BRACKET

**Figure 2.5.2.1.2**  
**Paving Brackets**



INPLACE SECTION REMOVAL



RECONSTRUCTED SECTION ③

- ① REMOVE PARAPET WALL AND PAVING BRACKET IN ITS ENTIRETY TO SOUND CONCRETE IN ACCORDANCE WITH THE SPECIAL PROVISIONS.
- ② MEMBRANE WATERPROOFING SYSTEM.
- ③ INCLUDED IN ITEM "RECONSTRUCT PAVING BRACKET AND WALL".

RECONSTRUCT PAVING BRACKET AND WALL

**Figure 2.5.2.1.3**  
**Paving Brackets**

**2.5.2.2 Piers**

[Future manual content]

**2.6 Construction Requirements**

MnDOT's general practices and guidelines for the construction of bridges are presented in MnDOT's *Bridge Construction Manual*, which can be accessed from the MnDOT Bridge Office web site.

Provide the required submittals and the qualifications of the individuals responsible for the preparation of falsework and other submittals in the contract documents for the project.

Falsework and forms are to be designed in accordance with the AASHTO *Guide Design Specifications for Bridge Temporary Works*. Falsework submittals must meet the requirements of Bridge Special Provision No. SB2018-2401.2.

Submittals describing proposed temporary shoring for works adjacent to railroad tracks require approval by the railroad.

Provide details of temporary shoring in the plans with consideration of the domestic availability of the sheeting materials. Frequently, showing the location of the sheeting and the minimum required section modulus is sufficient. However, designers should satisfy themselves that adequate clearances have been provided for at least one reasonable shoring scheme for staged construction projects. If more complex details are required, they must be provided in the plans. See Article 11.3.7 of this manual for more guidance.

**APPENDIX 2-A  
BRIDGE TYPE NUMBERS**

**MINNESOTA BRIDGE TYPE IDENTIFICATION NUMBER (3 characters)**

First Digit (Superstructure Material)	Second & Third Digits (Bridge Type)
1 Cast-In-Place Concrete	01 Beam Span
2 Cast-In-Place Concrete Continuous	02 Low Truss
3 Steel	03 High Truss
4 Steel Continuous	04 Deck Truss
5 Prestressed Pretensioned or Precast Concrete	05 Thru Girder
6 Prestressed Pretensioned Continuous Concrete	06 Deck Girder
7 Timber	07 Box Girder
8 Masonry	08 Rigid Frame
9 Wrought or Cast Iron	09 Slab Span
O Other	10 Slab Span-Voided
A Aluminum	11 Channel Span
P Prestressed Post-Tensioned	12 Arch
	13 Box Culvert
	14 Pipe Culvert (Round)
	15 Pipe Arch
	16 Long Span
	17 Tunnel
	18 Movable
	19 Other
	20 Double Tee
	21 Quad Tee
	22 Bulb Tee
	23 Suspension
	24 Tied Arch
	25 Cable Stay
	26 Inverted T

EXAMPLES	
BRIDGE TYPE	ID NUMBER
Precast Concrete Box Culvert	513
Simple Span Cast-In-Place Concrete Slab	109
Tunnel in Rock	017
Pretensioned Concrete Beam Span	501 approach span
Steel Continuous Beam Span	401 main span
Post-Tensioned Concrete Box Girder	P07

Note: A bridge may have different identification numbers for each span. Identify all span types accordingly.

**APPENDIX 2-B  
STANDARD ABBREVIATIONS**

**A**

AASHTO ..... American Association of State Highway and Transportation Officials  
 ABT. .... About  
 ABUT. .... Abutment  
 AADT ..... Annual Average Daily Traffic  
 ADTT ..... Average Daily Truck Traffic  
 ALT. .... Alternate  
 APPR. .... Approach  
 APPROX. .... Approximate (or Approximately)  
 ASSY. .... Assembly  
 AZ. .... Azimuth  
 @ ..... At

**B**

B.F. .... Back Face  
 BIT. .... Bituminous  
 B.M. .... Bench Mark  
 BM ..... Beam  
 BOT. .... Bottom  
 BR. .... Bridge  
 BRG. .... Bearing  
 BTWN. .... Between

**C**

C & G..... Curb and Gutter  
 C-I-P ..... Cast-In-Place  
 CL ..... Centerline  
 CL. (or CLR.)..... Clear  
 C.M.P. .... Corrugated Metal Pipe  
 COL. .... Column  
 COMP. .... Composite  
 CONC. .... Concrete  
 CONST. .... Construction  
 CONT. .... Continuous (or Continued)  
 C.S.A.H. .... County State Aid Highway  
 CU. .... Cubic  
 CULV. .... Culvert

**D**

D.C. .... Degree of Curve  
 DET. .... Detail  
 D.H.V. .... Design Hourly Volume  
 D.H.W. .... Design High Water  
 DIA. .... Diameter  
 DIAPH. .... Diaphragm  
 DL ..... Dead Load  
 DWL. .... Dowel

**E**

E. .... East  
 E.B.L. .... East Bound Lane(s)  
 E.F. .... Each Face  
 EA. .... Each  
 ELEV. (or EL.) ..... Elevation  
 EMBED. .... Embedment  
 ENGR. .... Engineer  
 EQ. .... Equal  
 EXP. .... Expansion

**F**

F. .... Fahrenheit  
 F.B.M. .... Foot Board Measure  
 F.F. .... Front Face  
 F.L. .... Flowline  
 FIN. .... Finished  
 FIX ..... Fixed  
 FT. .... Foot (or Feet)  
 FTG. .... Footing

**G**

G1 ..... Grade One  
 G2 ..... Grade Two  
 GA. .... Gage

**APPENDIX 2-B (Continued)**  
**STANDARD ABBREVIATIONS**

**H**

HCAADT ..... Heavy Commercial Annual  
Average Daily Traffic  
H.W. .... High Water  
HORIZ. .... Horizontal  
HWY. .... Highway

**I**

INPL. .... Inplace  
I.D. .... Inside Diameter

**J**

JCT. .... Junction  
JT. .... Joint

**K**

KWY. .... Keyway

**L**

L. .... Length of Curve  
LL ..... Live Load  
L.W. .... Low Water  
LB. .... Pound  
LIN. .... Linear  
LT. .... Left  
LONG. (or LONGIT.)..... Longitudinal

**M**

m ..... Meter  
mm ..... Millimeter  
M.B.M. .... Thousand Board Feet  
M.L. .... Main Line  
M.O. .... Maximum Offset  
MAX. .... Maximum  
MIN. .... Minimum  
MISC. .... Miscellaneous

**N**

N. (or NO.)..... North  
N.B.L. .... North Bound Lane(s)  
NO. .... Number

**O**

O.D. .... Outside Diameter

**P**

P.C. .... Point of curvature  
P.C.C. .... Point of compound Curve  
P.G. .... Profile Grade  
P.I. .... Point of Intersection  
P.O.C. .... Point on Curve  
P.O.T. .... Point on Tangent  
P.S.I. .... Pounds per Square Inch  
P.T. .... Point of Tangency  
PED. .... Pedestrian  
Pl ..... Plate  
PRESTR. .... Prestressed  
PROJ. .... Project (or Projection)  
PROV. .... Provision  
PT. .... Point

**R**

R. .... Radius  
R.O.W. .... Right of Way  
R.R. .... Railroad  
R.S.C. .... Rigid Steel Conduit  
RDWY. .... Roadway  
REINF. .... Reinforced (or Reinforcing/ment)  
REQ'D..... Required  
REV. .... Revised  
RT. .... Right

**S**

S. (or SO.) ..... South  
S.B.L. .... South Bound Lane(s)  
SEC. .... Section  
SDWK. .... Sidewalk  
SHLDR. .... Shoulder

**APPENDIX 2-B (Continued)**  
**STANDARD ABBREVIATIONS**

**S (cont.)**


---

SHT. ....	Sheet
SP. (or SPS.) ....	Spaces
SPA. ....	Spaced
SPEC. ....	Special (or Specification)
SPG. ....	Spacing
SQ. ....	Square
STA. ....	Station
STD. ....	Standard
STIFF. ....	Stiffener
STL. ....	Steel
STR. (or STRUC.) ....	Structure
SUBGR. ....	Subgrade
SUPER. ....	Superelevation
SUPERST. ....	Superstructure
SYM. ....	Symmetrical

**T**


---

T & B .....	Top and Bottom
T.H. ....	Trunk Highway
T.T.C. ....	Tangent to Curve
TAN. ....	Tangent
TWP. ....	Township
TYP. ....	Typical

**V**


---

V.C. ....	Vertical Curve
V.P.C. ....	Vertical Point of Curvature
V.P.I. ....	Vertical Point of Intersection
V.P.T. ....	Vertical Point of Tangency
VAR. ....	Varies
VERT. ....	Vertical

**W**


---

W. ....	West
W.B.L. ....	West Bound Lane(s)
W.C. ....	Wearing Course
W.P. ....	Working Point
W.W. ....	Wingwall

**Y**


---

YD. ....	Yard
----------	------

**APPENDIX 2-C  
STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES**

**a. NEW BRIDGES OTHER THAN CONCRETE BOX CULVERTS**

DESIGNED IN ACCORDANCE WITH 20\_\_ AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

HL-93 LIVE LOAD

DEAD LOAD INCLUDES 20 POUNDS PER SQUARE FOOT ALLOWANCE FOR FUTURE WEARING COURSE MODIFICATIONS

[Use on all roadway bridge projects. Insert current year of specifications.]

DESIGNED IN ACCORDANCE WITH 20\_\_ AND CURRENT INTERIM AASHTO LRFD GUIDE SPECIFICATIONS FOR THE DESIGN OF PEDESTRIAN BRIDGES

PEDESTRIAN LIVE LOAD = 0.090 KSF

\_\_\*\_\_ TRUCK VEHICULAR LIVE LOAD

[Use on all pedestrian bridge projects. Insert current year of specifications.]

[\* Insert "H-5" for deck widths between parapet/curb faces  $\leq$  10 ft. For deck widths between parapet/curb faces  $>$  10 ft., insert "H-10"]

**MATERIAL DESIGN PROPERTIES:**

**REINFORCED CONCRETE:**

$f'_c = 4$  KSI CONCRETE

$f_y = 60$  KSI PLAIN AND EPOXY COATED BARS

$f_y = 75$  KSI STAINLESS STEEL BARS

$n = 8$  FOR REINFORCEMENT BARS

[Use on all projects. Delete stainless steel bars when not included in bridge plan.]

**PRETENSIONED CONCRETE:**

$f'_c =$  \_\_ KSI CONCRETE

$f_{pu} = 270$  KSI LOW RELAXATION STRANDS

$n = 1$  FOR PRETENSIONING STRANDS

0.75  $f_{pu}$  FOR INITIAL PRESTRESS

[Coordinate  $f'_c$  with beam detail sheet. Use on bridges with pretensioned beams.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES (CONT'D)**

**a. NEW BRIDGES OTHER THAN CONCRETE BOX CULVERTS (CONT'D)**

POST-TENSIONED CONCRETE:

$f'_c = \underline{\hspace{1cm}}$  KSI CONCRETE

$f_{pu} = 270$  KSI LOW RELAXATION STRANDS

$n = 1$  FOR POST-TENSIONING STRANDS

$0.75 f_{pu}$  FOR INITIAL PRESTRESS

TOP OF DECK DESIGNED FOR ZERO TENSION UNDER SERVICE LOADS

[Use as required, e.g. post-tensioned slabs or box girders. Insert concrete strength.]

STRUCTURAL STEEL:

$F_y = 36$  KSI STRUCTURAL STEEL SPEC. 3306 (PAINTED)

$F_y = 50$  KSI STRUCTURAL STEEL SPEC. 3309 (PAINTED)

$F_y = 70$  KSI STRUCTURAL STEEL SPEC. 3317 (HIGH PERFORMANCE)  
(PAINTED)

[Use as required on bridges with steel components. Include description within parentheses as needed.]

WOOD:

$F_{bo} = \underline{\hspace{1cm}}$  KSI PILE CAPS

$F_{bo} = \underline{\hspace{1cm}}$  KSI SAWN STRINGERS AND TIMBER RAILS

$F_{bo} = \underline{\hspace{1cm}}$  KSI GLUED LAMINATED TIMBER RAILS

$F_{bo} = \underline{\hspace{1cm}}$  KSI GLUED LAMINATED STRINGERS

$F_{bo} = \underline{\hspace{1cm}}$  KSI GLUED LAMINATED DECK PANELS

$F_{bo} = \underline{\hspace{1cm}}$  KSI NAIL LAMINATED DECK PANELS

$F_{bo} = \underline{\hspace{1cm}}$  KSI RAIL POSTS

$F_{bo} = \underline{\hspace{1cm}}$  KSI ALL OTHER WOOD

[Use as required on bridges with wood components. Insert reference design values.]

DESIGN SPEED:

OVER =            MPH

[Use on all projects. Insert speed.]

UNDER =            MPH

[Use as required. Insert speed.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES (CONT'D)**

**a. NEW BRIDGES OTHER THAN CONCRETE BOX CULVERTS (CONT'D)**

DECK AREA = \_\_\_\_\_ SQUARE FEET

[Use on all projects. Insert area based on dimensions from coping to coping and begin bridge to end bridge.]

20\_\_ PROJECTED TRAFFIC VOLUMES

ROADWAY OVER      ROADWAY UNDER

\_\_\_\_\_ AADT      \_\_\_\_\_

\_\_\_\_\_ DHV      \_\_\_\_\_

\_\_\_\_\_ HCAADT      \_\_\_\_\_

[Use when provided on the Preliminary Plan. Insert appropriate values. Note that HCAADT is Heavy Commercial Annual Average Daily Traffic, which can be considered equivalent to the Average Daily Truck Traffic (ADTT) used by the AASHTO LRFD Bridge Design Specifications.]

HL-93 LRFR

BRIDGE OPERATING RATING FACTOR RF = \_\_\_\_\_

[Use on all projects. Insert rating factor.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**A. DESIGN DATA AND PROJECTED TRAFFIC VOLUMES (CONT'D)**

**b. PRECAST CONCRETE BOX CULVERTS**

DESIGNED IN ACCORDANCE WITH \_\_\_\_ AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

[For standard precast concrete box culvert designs, insert date of specifications that standards are based on, which can be found at the top of Bridge Standard Plans Figure 5-395.100(A) Precast Concrete Box Culvert - Basis of Design. For non-standard designs, insert current year of specifications.]

HL-93 LIVE LOAD

BARREL SPAN = \_\_\_\_\_

BARREL RISE = \_\_\_\_\_

BARREL LENGTH = \_\_\_\_\_

EST. MIN. FILL DEPTH (A) = \_\_\_\_\_

EST. MAX. FILL DEPTH (B) = \_\_\_\_\_

SKEW ANGLE = \_\_\_\_° \_\_\_\_' \_\_\_\_"

[Use on all precast concrete box culvert projects. Insert appropriate values. For estimated min. and max. fill depths, also show (A) and (B) in the box culvert elevation view to depict the min. and max. fill heights at the outside edge of the roadway shoulder.]

DESIGN SPEED = \_\_\_\_ MPH

AADT = \_\_\_\_\_

[Use on non-trunk highway precast box culvert projects.]

MATERIAL DESIGN PROPERTIES:

PRECAST REINFORCED CONCRETE:

$f'_c$  = \_\_ KSI CONCRETE

$f_y$  = 65 KSI WELDED WIRE REINFORCEMENT

$f_y$  = 60 KSI REINFORCEMENT BARS

[Use on all precast concrete box culvert projects. Insert concrete strength.]

HL-93 LRFR

BRIDGE OPERATING RATING FACTOR RF = \_\_\_\_\_

[Use on all precast concrete box culvert projects where the box culvert has been assigned a bridge number. For standard designs, insert 1.3 rating factor. For non-standard designs, insert calculated rating factor.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**B. CONSTRUCTION NOTES**

THE 20\_\_ EDITION OF THE MINNESOTA DEPARTMENT OF TRANSPORTATION "STANDARD SPECIFICATIONS FOR CONSTRUCTION" SHALL GOVERN.

[Use on all projects. Insert current date of specifications.]

SEE SPECIAL PROVISIONS FOR ALL XXXX.6XX SERIES PAY ITEMS FOR ADDITIONAL REQUIREMENTS.

[Use on all projects.]

THE BAR SIZES SHOWN IN THIS PLAN ARE IN U.S. CUSTOMARY DESIGNATIONS.

[Use on all projects.]

BARS MARKED WITH THE SUFFIX "E" SHALL BE EPOXY COATED IN ACCORDANCE WITH SPEC. 3301.

[Use on all projects.]

BARS MARKED WITH THE SUFFIX "S" SHALL BE STAINLESS STEEL IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

[Use as required.]

THE SUBSURFACE UTILITY INFORMATION IN THIS PLAN IS UTILITY QUALITY LEVEL \_\_\_\_\_. THIS UTILITY QUALITY LEVEL WAS DETERMINED ACCORDING TO THE GUIDELINES OF CI/ASCE 38-02, ENTITLED "STANDARD GUIDELINES FOR THE COLLECTION AND DEPICTION OF EXISTING SUBSURFACE UTILITY DATA".

[Use on all projects. Insert quality level shown in Preliminary Bridge Plan. When unknown, the default level is "D", which is the lowest level.]

THE PILE LOADS SHOWN IN THE PLANS AND THE CORRESPONDING NOMINAL PILE BEARING RESISTANCE ( $R_n$ ) WERE COMPUTED USING LRFD METHODOLOGY. PILE BEARING RESISTANCE DETERMINED IN THE FIELD SHALL INCORPORATE THE METHODS AND/OR FORMULAS DESCRIBED IN THE SPECIAL PROVISIONS.

[Use on all plans involving CIP or H piles.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**B. CONSTRUCTION NOTES (CONT'D)**

THE GIRDERS HAVE BEEN DESIGNED AND DETAILED WITHOUT DIAPHRAGMS. THE CONTRACTOR'S ENGINEER SHALL DESIGN, AND THE CONTRACTOR SHALL CONSTRUCT A TEMPORARY BRACING SYSTEM AND/OR A DECK FALSEWORK/FORMWORK SYSTEM. THE SYSTEM SHALL PROVIDE LATERAL AND ROTATIONAL STABILITY OF THE GIRDERS TO RESIST UNSYMMETRICAL CONCRETE AND CONSTRUCTION LOADS UNTIL THE DECK CONCRETE HAS ATTAINED A MINIMUM STRENGTH OF 2800 PSI.

[Use on all plans with pretensioned RB shapes or 27M I-beams where diaphragms are not used.]

INSTALL SETTLEMENT PLATE BEHIND EACH ABUTMENT. SEE GRADING PLANS.

[Use when specified in Foundation Recommendations.]

CONSTRUCTION OF EACH ABUTMENT SHALL NOT BE STARTED UNTIL THE APPROACH FILL AT THAT ABUTMENT HAS BEEN CONSTRUCTED TO THE FULL HEIGHT AND CROSS SECTION (AND ALLOWED TO SETTLE FOR \_\_\_\_\_ DAYS).

[Use when specified in Foundation Recommendations. Insert waiting period if required.]

**C. SIGNATURE BLOCK**

APPROVED \_\_\_\_\_  
STATE BRIDGE ENGINEER

DATE \_\_\_\_\_

[Use on all projects. Provide signature line in the title block on the General Plan and Elevation sheet.]

**D. DRAINAGE AND EROSION CONTROL**

RESTORE SIDE DITCHES AFTER PLACEMENT OF SLOPE PAVING TO PROVIDE DRAINAGE AS DIRECTED BY THE ENGINEER. RESTORATION COSTS SHALL BE INCLUDED IN PRICE BID FOR STRUCTURE EXCAVATION.

[Use this note on railroad underpasses.]

\_\_\_\_\_ PIPE TO BE PLACED UNDER GRADING PORTION OF CONTRACT.

[Use this note with combined Bridge and Roadway contracts only. Modify the notes to suit job requirements. Insert pipe description.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**E. EXCAVATION AND EARTHWORK**

QUANTITY OF STRUCTURE EXCAVATION FOR PAYMENT IS COMPUTED WITH THE ELEVATION SHOWN FOR EACH SUBSTRUCTURE UNIT AS THE UPPER LIMIT. EXCAVATION ABOVE THESE ELEVATIONS WILL BE PAID FOR UNDER THE GRADING PORTION OF THE CONTRACT.

[Use this note when rock and other type excavation will be encountered. Do not use this note when lump sum payment for structure excavation is used. Specify an elevation for top of exposed or buried rock and add the note "Average elevations of top of rock are assumed for estimated plan quantities."]

THE LOWER LIMITS OF STRUCTURE EXCAVATION CLASS E SHALL BE THE SAME AS THE UPPER LIMITS OF STRUCTURE EXCAVATION CLASS WE EXCEPT FOR ROCK EXCAVATIONS.

[Use as required.]

ROADWAY (OR CHANNEL) EXCAVATION WILL BE MADE BY OTHERS IN ADVANCE OF BRIDGE CONSTRUCTION.

[Not applicable on combined project.]

FOOTINGS SHALL BE KEYED INTO SOUND BEDROCK AS DIRECTED BY THE ENGINEER. TOP OF FOOTINGS SHALL HAVE A MINIMUM OF 1'-0" COVER.

[Use as required.]

CONTRACTOR SHALL DRESS SLOPES AND PLACE FILTER MATERIALS AND RIPRAP IN APPROXIMATE AREAS AS DIRECTED BY THE ENGINEER.

[Use as required.]

SUBCUT FOOTING A MINIMUM OF 2'-0" AND PLACE AGGREGATE BACKFILL (CONFORMING TO SPEC. 3149.2.E) PER SPEC. 2451 AS SHOWN. COMPACT TO 100% MAXIMUM DENSITY IN ACCORDANCE WITH SPEC 2105.3.F.1. SEE SPECIAL PROVISIONS.

[Use when a subcut for a spread footing is required.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**F. REINFORCEMENT**

**SPIRAL DATA**

    OUTSIDE DIAMETER \_\_\_\_\_

    HEIGHT \_\_\_\_\_

    PITCH \_\_\_\_\_

    SPIRAL ROD SIZE, PLAIN ROUND \_\_\_\_\_

    WEIGHT, EACH \_\_\_\_\_

    [Use for round columns that contain spiral reinforcement. Insert appropriate data.]

OUTSIDE DIAMETER OF DOWEL CIRCLE TO BE 2<sup>1</sup>/<sub>4</sub>" LESS THAN INSIDE DIAMETER OF SPIRAL.

[Use for round columns that contain spiral reinforcement. Where No. 10 and larger sized column vertical bars are used, increase the 2<sup>1</sup>/<sub>4</sub>" dimension where required to provide for a proper fit.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**G. FOUNDATIONS**

**a. SPREAD FOOTINGS ON SOIL**

_____ ABUTMENT SPREAD FOOTING LOAD DATA	
* _____ † _____ DESIGN BEARING PRESSURE	TONS/SQ FT
EFFECTIVE WIDTH B'	FT
_____ † _____ BEARING RESISTANCE $\phi_b \cdot Q_n$	TONS/SQ FT

\* BASED ON \_\_\_\_\_ LOAD COMBINATION.

PIER _____ SPREAD FOOTING LOAD DATA	
* _____ † _____ DESIGN BEARING PRESSURE	TONS/SQ FT
EFFECTIVE WIDTH B' (PERPENDICULAR TO PIER)	FT
EFFECTIVE LENGTH L' (PARALLEL TO PIER)	FT
_____ † _____ BEARING RESISTANCE $\phi_B \cdot Q_n$	TONS/SQ FT

\* BASED ON \_\_\_\_\_ LOAD COMBINATION.

[Use for spread footing foundations on soil. Insert data. Round loads to nearest 0.1 tons/sq ft.]  
 [ † Insert "SERVICE" or "FACTORED" based on governing limit state.]

**b. SPREAD FOOTINGS ON ROCK**

_____ ABUTMENT SPREAD FOOTING LOAD DATA	
* MAX FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* MIN FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
FACTORED BEARING RESISTANCE $\phi_b \cdot Q_n$	TONS/SQ FT

\* BASED ON \_\_\_\_\_ LOAD COMBINATION.

PIER _____ SPREAD FOOTING LOAD DATA	
* FTG CORNER 1 MAXIMUM FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* FTG CORNER 2 FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* FTG CORNER 3 FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
* FTG CORNER 4 FACTORED TRAPEZOIDAL DESIGN BEARING PRESSURE	TONS/SQ FT
FACTORED BEARING RESISTANCE $\phi_b \cdot Q_n$	TONS/SQ FT

\* BASED ON \_\_\_\_\_ LOAD COMBINATION.

[Use for spread footing foundations on rock. Define footing corners in bridge plan. Insert data. Round loads to nearest 0.1 tons/sq. ft.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**G. FOUNDATIONS (CONT'D)**

**c. LOADS FOR PILES**

_____ ABUTMENT COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD + EARTH PRESSURE	
FACTORED LIVE LOAD	
* FACTORED DESIGN LOAD	

\* BASED ON \_\_\_\_\_ LOAD COMBINATION.

PIER _____ COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD	
FACTORED LIVE LOAD	
FACTORED OVERTURNING	
* FACTORED DESIGN LOAD	

\* BASED ON \_\_\_\_\_ LOAD COMBINATION.

[Use for pile foundations. Insert data. Round loads to nearest 0.1 tons/pile.]

**d. ADDITIONAL LOAD TABLE FOR PILES WHERE DOWNDRAG OCCURS**

_____ ABUTMENT COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD + EARTH PRESSURE	
FACTORED DOWNDRAG	
*** FACTORED DEAD LOAD + EARTH PRESSURE + DOWNDRAG	

\*\*\* BASED ON \_\_\_\_\_ LOAD COMBINATION,  
 NOT INCLUDING TRANSIENT LOADS. ONLY USED FOR  
 COMPARISON WITH FACTORED STRUCTURAL  
 RESISTANCE. NOT TO BE USED FOR DRIVING.

PIER _____ COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD	
FACTORED OVERTURNING	
FACTORED DOWNDRAG	
*** FACTORED DEAD LOAD + OVERTURNING + DOWNDRAG	

\*\*\* BASED ON \_\_\_\_\_ LOAD COMBINATION,  
 NOT INCLUDING TRANSIENT LOADS. ONLY USED FOR  
 COMPARISON WITH FACTORED STRUCTURAL  
 RESISTANCE. NOT TO BE USED FOR DRIVING.

[Use for pile foundations where downdrag occurs. Use this table in addition to the standard load table shown above in c. Insert data. Round loads to nearest 0.1 tons/pile.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**G. FOUNDATIONS (CONT'D)**

**e. H-PILE RESISTANCE**

_____ ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE FOR H-PILES $R_n$ – TONS/PILE		
FIELD CONTROL METHOD	$\phi_{dyn}$	** $R_n$
MNDOT PILE FORMULA 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.60	
PDA	0.65	

\*\*  $R_n = (\text{FACTORED DESIGN LOAD}) / \phi_{dyn}$

_____ PIER REQUIRED NOMINAL PILE BEARING RESISTANCE FOR H-PILES $R_n$ – TONS/PILE		
FIELD CONTROL METHOD	$\phi_{dyn}$	** $R_n$
MNDOT PILE FORMULA 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.60	
PDA	0.65	

\*\*  $R_n = (\text{FACTORED DESIGN LOAD}) / \phi_{dyn}$

[Use for H-pile foundations. Insert data. Round loads to nearest 0.1 tons/pile.]

**f. CIP CONCRETE PILE RESISTANCE**

_____ ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES $R_n$ – TONS/PILE		
FIELD CONTROL METHOD	$\phi_{dyn}$	** $R_n$
MNDOT PILE FORMULA 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	
PDA	0.65	

\*\*  $R_n = (\text{FACTORED DESIGN LOAD}) / \phi_{dyn}$

_____ PIER REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES $R_n$ – TONS/PILE		
FIELD CONTROL METHOD	$\phi_{dyn}$	** $R_n$
MNDOT PILE FORMULA 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	
PDA	0.65	

\*\*  $R_n = (\text{FACTORED DESIGN LOAD}) / \phi_{dyn}$

[Use for cast-in-place concrete pile foundations. Insert data. Round loads to nearest 0.1 tons/pile.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**G. FOUNDATIONS (CONT'D)**

**g. OTHER**

PILE NOTES [Use for substructure with test piles]

\_\_\_\_\_ TEST PILES \_\_\_\_\_ FT. LONG  
 \_\_\_\_\_ PILES EST. LENGTH \_\_\_\_\_ FT.  
 \_\_\_\_\_ PILES REQ'D FOR \_\_\_\_\_

[Use for projects with piles. Insert number of piles, pile type, pile length, and substructure name.]

PILE NOTES [Use for substructure without test piles]

\_\_\_\_\_ PILES \_\_\_\_\_ FT. LONG REQ'D. FOR \_\_\_\_\_

[Use for projects with piles. Insert number of piles, pile type, pile length, and substructure name.]

PILE NOTES [substructure with special pay items]

\_\_\_\_\_ PILES EST. LENGTH \_\_\_\_\_ FT. REQ'D FOR \_\_\_\_\_

[Use for projects with piles. Insert number of piles, pile type, pile length, and substructure name.]

**GENERAL PILE NOTES**

PILE SPACING SHOWN IS AT BOTTOM OF \_\_\_\_\_\*.

[Use for projects with piles.]

[\* Insert "FOOTING" or "ABUTMENT"]

PILES MARKED THUS (O-> , H-> ) TO BE BATTERED \_\_\_\_\_ PER FOOT IN DIRECTION SHOWN.

[Use for all battered piling.]

PILES TO BE HP - \_\_\_\_\_.

[Use with all steel H piling.]

PILES TO HAVE A NOMINAL DIAMETER OF \_\_\_\_\_ (AND A THICKNESS OF \_\_\_\_\_").

[Use with all CIP piles. Insert pile diameter and also wall thickness when specified in Foundation Recommendations.]

FOR PILE SPLICE DETAILS SEE DETAIL \_\_\_\_\_\*

[Use for projects with piles.]

[\* For CIP piles, insert "B201". For H-piles, insert "B202".]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**G. FOUNDATIONS (CONT'D)**

DRIVE TEST PILES TO ESTIMATED FOUNDATION PILE LENGTH. IF DESIGN BEARING IS NOT OBTAINED, WAIT 24 HOURS AND THEN PERFORM REDRIVE. IF DESIGN BEARING IS NOT OBTAINED AFTER REDRIVE, CONTINUE DRIVING TO FULL TEST PILE LENGTH.

[Use when pile redriving is specified in Foundation Recommendations.]

DENOTES REACTION PILE TO BE INCLUDED IN ITEM "\_\_\_\_\_ TEST PILE \_\_\_ FT. LONG \_\_\_\_". ANY PILE SPLICES MUST BE WELDED ON REACTION PILES. SEE SPECIAL PROVISIONS.

[Use for pile load test. Insert pile type, length, and size.]

DENOTES STATIC LOAD TEST PILE. DRIVE TO \_\_\_ FT. LONG. TO BE INCLUDED IN ITEM "PILE LOAD TEST TYPE \_\_\_". USE STATIC LOAD TEST PILE AS STRUCTURAL SUPPORT PILE AFTER LOAD TEST IS COMPLETE.

[Use for pile load test. Insert length to drive and pile type.]

FOUNDATION PILES SHALL NOT BE DRIVEN UNTIL RESULTS OF STATIC LOAD TEST ARE PROVIDED TO THE ENGINEER. THE ENGINEER WILL AUTHORIZE PILE LENGTHS NO LATER THAN 3 DAYS AFTER RECEIPT OF LOAD TEST RESULTS.

[Use for pile load test.]

**H. STEEL MATERIALS, FABRICATION, AND ERECTION**

DIMENSIONS SHOWN ARE BASED ON AN AMBIENT TEMPERATURE OF 45°F. FABRICATOR TO ADJUST FOR TEMPERATURE AT FABRICATION.

[Include this note on the framing plan.]

ALL STRUCTURAL STEEL SHALL CONFORM TO SPEC 3309 UNLESS OTHERWISE NOTED.

[Use as required.]

SHEAR STUDS ON THE TOP FLANGE OF THE GIRDER SHALL BE INSTALLED IN THE FIELD.

[Use as required.]

THE MAXIMUM RESIDUAL CAMBER IN SPAN \_\_\_\_\_ IS \_\_\_ INCHES AT THE \_\_\_\_\_ POINT OF THE SPAN.

[Include this note on camber diagram sheet when residual camber has been included. Insert span number, residual camber, and location.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**H. STEEL MATERIALS, FABRICATION, AND ERECTION (CONT'D)**

THERE IS NO RESIDUAL CAMBER IN THESE BEAMS.

[Include this note on camber diagram sheet when residual camber has not been included.]

CHORD LINE IN CAMBER DIAGRAM IS A STRAIGHT LINE FROM END TO END OF BEAM SEGMENT AT \_\_\_\_\_\*\_\_\_\_\_.

[Include this note on camber diagram sheet of steel girder bridges]

[\* For plate girders, insert "TOP OF BEAM WEB". For rolled beams, insert "BOTTOM OF TOP FLANGE".]

CAMBER DIAGRAM SHOWN IS FOR BEAMS IN UNLOADED POSITION AND PROVIDES FOR ALL DEAD LOAD DEFLECTIONS (AND RESIDUAL CAMBER).

[Include this note on camber diagram sheet of steel girder bridges. Include residual camber portion of note when used in the beams.]

LINE ASSEMBLY PER SPEC. 2471 WILL BE REQUIRED FOR BEAM SPLICES. THE SECTION TO BE LINE ASSEMBLED SHALL BE FROM \_\_\_\_\_ TO \_\_\_\_\_ .

[Use on bridges where girders were designed using line girder analysis. Check with the Structural Metals Unit; Abutment to abutment if < 300 ft. Three adjacent points of support if > 300 ft.]

FULL ASSEMBLY WILL BE REQUIRED PER SPEC. 2471.

[The use of full assembly should be considered for extremely complicated curved, superelevated structures (i.e. grid or 3D analysis used for design). Check with the Structural Metals Unit and Fabrication Methods Unit.]

PRIOR TO POURING DECK, SNUG TIGHTEN ONLY BOLTS IN THE END DIAPHRAGMS. AFTER POURING ALL DECK CONCRETE, FULLY TIGHTEN END DIAPHRAGM BOLTS.

[Use on all skewed steel bridges.]

PROVIDE OVERSIZED HOLES IN END DIAPHRAGM CONNECTIONS.

[Use on all skewed steel bridges.]

DETAIL AND FABRICATE STEEL MEMBERS FOR A NO-LOAD FIT.

[Use on all steel girder bridges.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**H. STEEL MATERIALS, FABRICATION, AND ERECTION (CONT'D)**

FOR WELDED FLANGE SPLICES, SEE SPEC 2471.3.F.1.a.

[Use drawings instead of note only if different than AASHTO/AWS D1.5 Bridge Welding Code 2010 Fig. 2.7 or 2.8.]

WEB PLATES SHALL BE FURNISHED IN AVAILABLE MILL LENGTHS AND WIDTHS WITH A MINIMUM NUMBER OF WEB SPLICES. LOCATION OF SPLICES SHALL BE SUBJECT TO THE APPROVAL OF THE ENGINEER AND SHALL BE A MINIMUM OF 1'-0" FROM STIFFENERS OR FLANGE SPLICES.

[Use on all steel plate girder bridges.]

BEARING STIFFENERS AT ABUTMENTS SHALL BE VERTICAL. BEARING STIFFENERS AT PIERS AND DIAPHRAGM CONNECTION STIFFENERS SHALL BE PERPENDICULAR TO FLANGE. ENDS OF BEAMS SHALL BE VERTICAL.

[Use for grades greater than 3% on plate girder bridges or skews greater than 20°.]

BEARING STIFFENERS, DIAPHRAGM CONNECTION STIFFENERS, AND ENDS OF BEAMS SHALL BE PERPENDICULAR TO FLANGE.

[Use for rolled beams or grades  $\leq 3\%$ .]

ROWS OF SHEAR CONNECTORS SHALL BE ALIGNED PARALLEL TO THE TRANSVERSE SLAB REINFORCEMENT BARS.

[Use on all steel bridges with shear studs.]

SHEAR CONNECTORS TO PROJECT A MINIMUM OF 2" INTO DECK STRUCTURAL SLAB. IN NO CASE SHALL SHEAR CONNECTORS PROJECT CLOSER THAN   \*  " TO TOP OF DECK STRUCTURAL SLAB. ENGINEER TO FIELD VERIFY BEAM ELEVATION AND AUTHORIZE STUD LENGTH.

[Use on all steel bridges with shear studs.]

[\*Insert "1" for decks with 2" wearing course. Insert "3" for monolithic decks.]

FLANGE PLATES FOR BEAMS SHALL BE CUT TO PROPER CURVATURE.

[Use this note when the minimum radius of curvature requirements of AASHTO LRFD Bridge Construction Specifications, Article 11.4.12.2.2 are not met.]

ALL BOLTED CONNECTIONS SHALL BE MADE WITH  $\frac{7}{8}$ " DIAMETER A325 BOLTS, EXCEPT AS NOTED.

[Use as required.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**H. STEEL MATERIALS, FABRICATION, AND ERECTION (CONT'D)**

ELEVATIONS SHOWN AT FIELD SPLICES ARE THEORETICAL ELEVATIONS FURNISHED AS A GUIDE FOR ERECTION. ELEVATIONS ARE GIVEN AT TOP OF TOP FLANGE SPLICE PLATE AND INCLUDE DEFLECTIONS FROM WEIGHT OF BEAM AND DIAPHRAGM.

[Use as required.]

DEFLECTIONS SHOWN ARE FOR WEIGHT OF SLAB, CONCRETE OVERLAY, \_\_\_\_\_.  
NEGATIVE SIGN INDICATES UPLIFT.

[Use as required. Insert barrier, parapet, metal railing, sidewalk, median, etc. as needed. Do not include the weight of steel beams or future wearing course.]

SHEAR CONNECTORS TO CONFORM TO SPEC. 3391.

[Use on all steel bridges with shear studs.]

**I. CONCRETE PLACEMENTS**

REMOVE ALL NON-GALVANIZED AND NON-EPOXY COATED FERROUS METAL, EXCLUDING SHEAR STUDS, TO WITHIN ½" OF THE TOP FLANGE PRIOR TO CASTING THE DECK.

[Use on all deck-on-beam type bridges. Locate note on the Transverse Section Deck Reinforcement sheet in the bridge plan.]

MAKE ¾" WIDE x 1" DEEP SAW CUT IN STRUCTURAL SLAB (AND CONCRETE WEARING COURSE) OVER CENTERLINE OF PIERS AS SOON AS THE CUTTING CAN BE DONE WITHOUT RAVELING THE CONCRETE. APPLY TYPE B POLYSTYRENE TO TIPS OF FLANGES THAT PROJECT PAST CENTERLINE OF PIER. SEAL JOINT PER SPEC. 3725.

[Use on prestressed concrete beam bridges with double diaphragms and slab continuous over piers. Saw cut both structural slab and concrete wearing course. See Figure 9.2.1.10 in this manual for detail.]

TOP OF SLAB UNDER BARRIER IS LEVEL. THE BOTTOM OF SLAB CONTINUES AT THE SAME SLOPE AS THE ROADWAY.

[Use for slab type bridges. Include note on superstructure sheet that contains cross-section.]

CAST COUNTER WEIGHT AT LEAST 48 HOURS IN ADVANCE OF PLACING DECK SLAB.

[Use as required.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**I. CONCRETE PLACEMENTS (CONT'D)**

CONSTRUCT BARRIER SUCH THAT ANGLE BETWEEN ROADWAY SURFACE AND BACK FACE OF BARRIER IS NOT GREATER THAN 90°.

[Use for barrier on the high side of bridges where superelevation is > 2%.]

**J. WELDED STEEL BEARING ASSEMBLIES**

STRUCTURAL STEEL SHALL CONFORM TO SPEC. 3306 EXCEPT AS NOTED.

[Use as required.]

SHIMS TO BE INCLUDED IN PRICE BID FOR BEARING ASSEMBLIES.

[Add to B Detail if shims are used.]

PINS AND ROLLERS SHALL CONFORM TO SPEC. 2471.3.D.4.

[Use as required.]

PINS SHALL BE COLD FINISHED ALLOY BAR STEEL PER SPEC. 3314 TYPE II.

[For pins 5" or less where pin is not made from a larger diameter stock.]

PINS SHALL BE HOT ROLLED ALLOY BAR STEEL PER SPEC. 3313 TYPE II.

[For pins over 5" where pin will be made from a larger diameter stock.]

PINTLES SHALL CONFORM TO SPEC. 3309.

[Use as required.]

LUBRICATED BRONZE BUSHINGS SHALL CONFORM TO SPEC. 3329.

[Use as required.]

ALL WELDED BEARING ASSEMBLIES SHALL BE ANNEALED AFTER WELDING. PIN HOLES AND TOP AND BOTTOM PLATES SHALL BE FINISHED AFTER ANNEALING.

[For welded rockers and bolsters.]

PINS AND PIN HOLES SHALL BE COATED IN THE SHOP WITH A HEAVY PROTECTIVE GREASE. PRIOR TO ERECTION, THE PINS AND PIN HOLES SHALL BE CLEANED AND COATED WITH AN APPROVED GREASE.

[Use as required.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**J. WELDED STEEL BEARING ASSEMBLIES (CONT'D)**

PROVIDE A TEMPLATE DEMONSTRATING THAT ANCHOR RODS WILL HAVE 2 INCHES CLEAR DISTANCE TO ALL REINFORCEMENT. REFER TO SPEC. 2472.3.C.1 FOR MORE DETAILED INSTRUCTION.

[Include this note with anchor rod placement details.]

\_\_\_ DENOTES ELASTOMERIC BEARING PAD, TYPE \_\_\_. SEE DETAIL B305.

\_\_\_ DENOTES FIXED CURVED PLATE BRG. ASSEMBLY, TYPE \_\_\_. SEE DETAIL B310.

\_\_\_ DENOTES EXPANSION CURVED PLATE BRG. ASSEMBLY, TYPE \_\_\_. SEE DETAIL B311.

[Include appropriate notes(s) on framing plan when elastomeric bearing pads are used. Insert bearing designation and type number.]

PLAN BRIDGE SEAT ELEVATIONS ARE BASED ON AN ASSUMED DISC BEARING HEIGHT OF \_\_\_. DETERMINE FINAL BRIDGE SEAT ELEVATIONS BASED ON ACTUAL HEIGHT OF DISC BEARING ASSEMBLIES. ANY REQUIRED ADJUSTMENTS TO THE BRIDGE SEAT ELEVATIONS SHALL BE MADE BY THE CONTRACTOR AT NO COST TO MnDOT. BRIDGE SEAT PEDESTALS SHALL NOT BE LESS THAN 3" TALL.

[To be used when disc bearings are provided. Include this note on substructure plan sheet where bridge seat elevations are shown. Insert estimated height of disc bearing assembly.]

**K. CUTTING AND REMOVAL OF OLD CONCRETE**

HATCHED AREAS INDICATE CONCRETE TO BE REMOVED.

[Use as required.]

NO CUTTING WILL BE PERMITTED UNTIL THE CUTTING LIMITS HAVE BEEN OUTLINED BY THE CONTRACTOR AND APPROVED BY THE ENGINEER. REMOVAL AND RECONSTRUCTION SHALL CONFORM TO SPEC. 2433.

[Use as required.]

**L. JOINTS AND JOINT SEALER**

PLACE CONTROL JOINT IN SIDEWALK (AND MEDIAN) OVER CENTERLINE OF PIER. FINISH TOP OF SIDEWALK (AND MEDIAN) JOINTS WITH 1/4" RADIUS EDGER, AND VERTICAL EDGES WITH 1/2" V STRIPS. BREAK BOND AT JOINT BY APPROVED METHOD. NO REINFORCEMENT THROUGH JOINT.

[Use for concrete sections less than 12" in height.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**L. JOINTS AND JOINT SEALER (CONT'D)**

1½" TYPE B POLYSTYRENE BETWEEN APPROACH PANEL AND WINGWALL TO BE INCLUDED IN GRADING PLAN.

[Use as required.]

SUPERSTRUCTURE DIMENSIONS ARE BASED ON THE EXPANSION JOINT DIMENSION AT 90° F.

[Use when superstructure details show more than one expansion joint dimension for different temperatures.]

**M. TIMBER BRIDGES**

CONSTRUCTION REQUIREMENTS PER SPEC. 2403.

[Use as required.]

ALL TIMBER PILING TO MEET REQUIREMENTS OF SPEC. 3471.

ALL HARDWARE TO BE GALVANIZED PER SPEC. 3392.

THREADS ON ALL BOLTS TO BE UPSET AFTER INSTALLATION.

[Use as required.]

ALL WOOD TO BE FINISHED AS SHOWN IN THE BILL OF MATERIALS.

[Use as required.]

TOP OF WING PILE WHICH PROJECTS OUTSIDE OF WING CAP SHALL BE SHAPED TO A 45° SLOPE.

[Use as required.]

TREAT TOPS OF WING PILES PER SPEC. 2403.3.E. SEE SPEC. 3491 FOR PRESERVATION REQUIREMENTS.

[Use as required.]

FILL IN BACK OF ABUTMENT IS NOT TO BE PLACED UNTIL AFTER SUPERSTRUCTURE HAS BEEN COMPLETED.

[Use as required.]

FASTEN BACKING TO ABUTMENT PILES WITH TWO 60D NAILS AT EACH INTERSECTION.

[Use as required.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**M. TIMBER BRIDGES (CONT'D)**

BOLT PROJECTIONS EXCEEDING 1" SHALL BE CUT OFF. REPAIR END OF BOLT BY PAINTING WITH AN APPROVED ZINC-RICH PRIMER.

[Use as required.]

TIMBER WINGWALL PILES ARE TO BE INSTALLED TO THE LENGTH SHOWN IN THE PLANS.

[Use as required.]

ALL WOOD SHALL BE PRESERVATIVE TREATED IN ACCORDANCE WITH SPEC. 3491.

[Use as required.]

ALL WOOD THAT IS CUT OR DRILLED IN THE FIELD SHALL BE FIELD TREATED IN ACCORDANCE WITH SPEC. 2403.3.E.

[Use as required.]

ALL PLANKS FOR PREFAB PANELS SHALL BE DOUGLAS FIR-LARCH GRADE \_\_\_\_ (F<sub>bo</sub> = \_\_\_\_ KSI).

[Use as required. Insert grade and reference design value.]

FOR GLULAM RAIL ELEMENTS SEE PLAN SHEET \_\_\_\_ .

[Use as required. Insert sheet number.]

RAIL POSTS, CURBS, SCUPPERS, AND RAIL SPACER BLOCKS SHALL BE DOUGLAS FIR-LARCH GRADE \_\_\_\_ (MIN. F<sub>bo</sub> = \_\_\_\_ KSI).

[Use as required. Insert grade and reference design value.]

ABUTMENT AND PIER CAPS SHALL BE DOUGLAS FIR-LARCH GRADE \_\_\_\_ POSTS AND TIMBERS (F<sub>bo</sub> = \_\_\_\_ KSI). ALL OTHER LUMBER SHALL HAVE MIN. F<sub>bo</sub> = \_\_\_\_ KSI.

[Use as required. Insert grade and reference design values.]

PILE PLATE ASSEMBLY SHALL BE SHOP PAINTED AFTER FABRICATION IS COMPLETED.

PAINT PIER PILES, PLATES AND RINGS PER SPEC. 2452.3.J.1

AREAS OF PILES, PLATES AND RINGS BURNED DURING WELDING SHALL BE CLEANED AND SPOT COATED WITH ZINC-RICH PRIMER BEFORE FIELD PAINTING IS STARTED.

[Use as required.]

**APPENDIX 2-C (Continued)**  
**STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**M. TIMBER BRIDGES (CONT'D)**

ALL STRUCTURAL STEEL SHALL CONFORM TO SPEC 3306 UNLESS OTHERWISE NOTED.

[Use as required.]

ALL STRUCTURAL STEEL SHALL BE GALVANIZED PER SPEC. 3394.

[Use as required.]

**N. MISCELLANEOUS**

F.F. DENOTES FRONT FACE.

B.F. DENOTES BACK FACE.

E.F. DENOTES EACH FACE.

[Use as required.]

THE CONTRACTOR SHALL MAKE FIELD MEASUREMENTS AS NECESSARY PRIOR TO FABRICATION OF THE \_\_\_\_\_ TO ASSURE PROPER FIT IN THE FINAL WORK.

[Use when not otherwise referenced to Spec. 2433. Insert component to be fabricated.]

BEAM LENGTH DIMENSIONS ARE SLOPED LENGTHS.

[Use where necessary for proper fit for prestressed beams.]

USE GREASE FROM THE "APPROVED/QUALIFIED PRODUCT LIST FOR BRIDGE PRODUCTS, BRIDGE GREASE".

[Use for dowel bar assemblies.]

ALL MATERIAL TO BE EPOXY COATED. MATERIAL AND PLACEMENT TO BE INCLUDED IN PRICE BID FOR "DOWEL BAR ASSEMBLY (EPOXY COATED)".

[Use for dowel bar assemblies.]

\_\_\_\_ PASSAGE BENCH AT EL. \_\_\_\_\_. SURFACE WITH CLASS V AGGREGATE TO PROVIDE SMOOTH SURFACE. INCLUDED IN ITEM "RANDOM RIPRAP CLASS \_\_\_\_".

[Use when a passage bench is required. Insert appropriate values.]

"X" DENOTES X END OF BEAM.

[Include on framing plan for pretensioned concrete beam bridges.]

**APPENDIX 2-C (Continued)****STANDARD PLAN NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

**N. MISCELLANEOUS (CONT'D)**

PROVIDE 1/8 IN. 60 DUROMETER PLAIN ELASTOMERIC PAD OR PREFORMED FABRIC PAD MEETING AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATION ARTICLE 18.10. WAIVE THE SAMPLING AND TESTING REQUIREMENTS UNDER SPEC 3741, "ELASTOMERIC BEARING PADS," AND AASHTO M251.

[Use as required.]

## APPENDIX 2-D

## STANDARD SUMMARY OF QUANTITIES NOTES

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

PAYMENT FOR \_\_\_\_\_ INCLUDED IN ITEM "\_\_\_\_\_".

[For minor items with small quantities that are not listed as pay items (joint filler, nameplate, etc.). Fill in blanks with description and appropriate pay item (e.g. for joint filler: Payment for joint filler included in item "Structural Concrete (3B52))".]

INCLUDED IN WEIGHT OF "STRUCTURAL STEEL (33\_\_)".

[Miscellaneous steel quantities (protection angle, etc.).]

DOES NOT INCLUDE TEST PILES.

[Use when piling quantities are listed.]

INCLUDES SLAB, END DIAPHRAGM, MEDIAN BARRIER, SIDEWALK, AND RAILING REINFORCEMENT.

[Add to epoxy coated reinforcement bar totals. Remove components that do not apply.]

"CONCRETE WEARING COURSE (3U17A)" INCLUDES \_\_\_\_\_ SQUARE FEET FOR BRIDGE APPROACH PANELS.

[Use when the item as listed in the Summary of Quantities for Superstructure is paid for on a square foot basis. Insert area.]

PAYMENT FOR BEARINGS INCLUDED IN ITEM "BEARING ASSEMBLY" PER EACH.

[Use as required.]

PAYMENT FOR ANCHORAGES INCLUDED IN ITEM "ANCHORAGES TYPE REINF BARS" PER EACH.

[Use as required.]

PAYMENT FOR ANCHORAGES INCLUDED IN ITEM "GROUTED REINFORCEMENT BARS" PER EACH.

[Use as required.]

PAYMENT FOR THREADED COUPLERS INCLUDED IN ITEM "COUPLERS (REINFORCEMENT BARS) T-\_\_\_" PER EACH.

[Use as required. Specify bar size.]

MEMBRANE WATERPROOFING SYSTEM PER SPEC 2481.3.B TO BE INCLUDED IN ITEM "STRUCTURAL CONCRETE (\_\_\_\_)".

[Use as required. Insert concrete mix.]

**APPENDIX 2-D (Continued)****STANDARD SUMMARY OF QUANTITIES NOTES**

Use standard notes that are relevant to the project. Text found in brackets [ ] next to a standard note provides guidance on its use and should not be included in the plan.

PAYMENT FOR SHEAR CONNECTORS INCLUDED IN ITEM "SHEAR STUDS".

[Use as required.]

SEE DRAINAGE SYSTEM TYPE (B910).

[Use as required.]

PILE REDRIVING AND PILE ANALYSIS FOR THE REACTION PILES AND STATIC LOAD TEST PILE SHALL BE INCLUDED IN PRICE BID FOR "PILE LOAD TEST TYPE \_\_\_".

[Use as required. Insert type number.]

APPENDIX 2-E  
CONVERSION FROM INCHES TO DECIMALS OF A FOOT

	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"
0	.0000	.0833	.1667	.2500	.3333	.4167	.5000	.5833	.6667	.7500	.8333	.9167
1/16	.0052	.0885	.1719	.2552	.3385	.4219	.5052	.5885	.6719	.7552	.8385	.9219
1/8	.0104	.0938	.1771	.2604	.3438	.4271	.5104	.5938	.6771	.7604	.8438	.9271
3/16	.0156	.0990	.1823	.2656	.3490	.4323	.5156	.5990	.6823	.7656	.8490	.9323
1/4	.0208	.1042	.1875	.2708	.3542	.4375	.5208	.6042	.6875	.7708	.8542	.9375
5/16	.0260	.1094	.1927	.2760	.3594	.4427	.5260	.6094	.6927	.7760	.8594	.9427
3/8	.0313	.1146	.1979	.2813	.3646	.4479	.5313	.6146	.6979	.7813	.8646	.9479
7/16	.0365	.1198	.2032	.2865	.3698	.4531	.5365	.6198	.7032	.7865	.8698	.9531
1/2	.0417	.1250	.2083	.2917	.3750	.4583	.5417	.6250	.7083	.7917	.8750	.9583
9/16	.0469	.1302	.2135	.2969	.3802	.4635	.5469	.6302	.7135	.7969	.8802	.9635
5/8	.0521	.1354	.2188	.3021	.3854	.4688	.5521	.6354	.7188	.8021	.8854	.9688
11/16	.0573	.1406	.2240	.3073	.3906	.4740	.5573	.6406	.7240	.8073	.8906	.9740
3/4	.0625	.1458	.2292	.3125	.3958	.4792	.5625	.6458	.7292	.8125	.8958	.9792
13/16	.0677	.1510	.2344	.3177	.4010	.4844	.5677	.6510	.7344	.8177	.9010	.9844
7/8	.0729	.1563	.2396	.3229	.4063	.4896	.5729	.6563	.7396	.8229	.9063	.9896
15/16	.0781	.1615	.2448	.3281	.4115	.4948	.5781	.6615	.7448	.8281	.9115	.9948
0	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"

CONVERSION TABLE  
INCHES TO DECIMALS OF A FOOT

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### **3. LOADS AND LOAD FACTORS**

The loads section of the AASHTO LRFD Specifications is greatly expanded over that found in the Standard Specifications. This section will present applicable loads and provide guidance to MnDOT's practice for the application of these loads.

#### **3.1 Load Factors and Combinations [3.4.1]**

The standard load combinations for LRFD design are presented in LRFD Table 3.4.1-1.

Several of the loads have variable load factors (e.g.,  $\gamma_p$ ,  $\gamma_{TG}$ ,  $\gamma_{SE}$ ). The load factors for permanent loads ( $\gamma_p$ ) typically have two values, a maximum value and a minimum value. When analyzing a structure it will often be necessary to use both values. The objective is to envelope the maximum load effects on various elements for design. A box culvert structure illustrates the use of both values. When determining the moment in the top slab of a culvert, the maximum load factor is used on the vertical earth loads, while the minimum load factor is used on the lateral or horizontal earth loads. The situation reverses when determining the moments in the wall of a culvert. A minimum load factor is used on the vertical earth loads and a maximum value is used on the horizontal earth loads.

When assembling load combinations, do not use more than one load factor for any load component. For example, when checking uplift, a load factor of 0.90 or 1.25 should be used for the dead load on all spans. Designers should not try to use 0.9 on the span adjacent to the uplift point and 1.25 on the next span.

Designers must ensure that structures have been checked for adequacy in carrying all appropriate load combinations at all construction stages. For example, check a high parapet abutment for any permissible construction case in addition to the final condition. The abutment may be completely constructed prior to placement of the beams (a case which maximizes the horizontal earth pressure load with a minimum of vertical load) or the abutment could be constructed such that the superstructure is completed prior to backfilling (a case which maximizes vertical load without horizontal earth pressure load). Designers are to investigate both cases. For complex structures, designers are responsible for providing one workable construction sequence in the bridge plan and checking for adequacy at all the construction stages. If the contractor proposes a different construction sequence, the contractor is responsible for confirming structure adequacy at all the construction stages.

**Load Combinations**

The load factors and the combination of different load components presented in LRFD Table 3.4.1-1 have been calibrated to produce structures with more uniform reliability than that offered with Standard Specification designs. The Extreme Event I load combinations will rarely control in Minnesota. Note that designs must also consider the load combinations for construction loading.

**Strength I:** Basic load combination used to determine the flexural and shear demands without wind.

**Strength II:** Basic load combination used to determine the flexural and shear demands of a structure subject to a permit vehicle or a special design vehicle specified by the owner. MnDOT does not typically use special vehicles for design. See Article 3.4 for more information.

**Strength III:** Load combination used to determine flexural and shear demands that include a design wind based on a 3-second gust wind speed of 115 mph.

**Strength IV:** Load combination relating to very high dead load to live load force effect ratios. Use the following modified Strength IV load combination, given in AASHTO LRFD Article C3.4.1:

**[C3.4.1]**

$$1.4DC + 1.5DW + 1.45LL$$

Note that Strength IV only applies to superstructures. It does not apply to investigation of construction stages, substructures, retaining walls, or bearings.

**Strength V:** Load combination corresponding to normal vehicular use of the bridge concurrent with a design wind based on a 3-second gust wind speed of 80 mph.

**Extreme Event I:** Load combination including earthquake effects. Earthquake analysis is typically not performed.

**Extreme Event II:** Load combination corresponding to ice loads, collision loads, and certain hydraulic events with a reduced vehicular live load. This combination is used for barrier design, deck overhang design, and pier design per the pier protection policy found elsewhere in this manual.

**Service I:** Load combination used for the design of many elements. It is used for service load stress checks (prestressed concrete), deflection checks, crack control checks in reinforced concrete, etc.

**Service II:** Load combination used to check yielding and connections in steel structures.

**Service III:** Load combination used to check outer fiber tension stresses and web principal stresses in prestressed concrete structures.

**Fatigue I:** Load combination used for the design of structures subject to repetitive live load. It is used for checking infinite load-induced fatigue life.

**Fatigue II:** Load combination used for the design of structures subject to repetitive live load. It is used for checking finite load-induced fatigue life.

**[3.4.2]**

**Construction:** All appropriate load combinations must be considered by designers for construction loads. Use the load factors given in AASHTO LRFD Article 3.4.2 for construction loads.

**3.2 Load Modifiers**  
**[1.3.3, 1.3.4, 1.3.5]**

For most structures, each of the load modifiers will be 1.00. For a limited number of bridges, load modifiers with values different from 1.00 need to be used. Table 3.2.1 summarizes MnDOT’s policy for load modifiers.

Note that load modifiers apply only to the strength limit state. For all other limit states, use a value of 1.00 for all load modifiers. Load modifiers need not be applied to construction load cases.

**Table 3.2.1**  
**Standard MnDOT Load Modifiers**

Modifier	Value	Condition
Ductility ( $\eta_D$ )	1.00	Steel structures, timber bridges, ductile concrete structures
	1.05	Non-ductile concrete structures
Redundancy ( $\eta_R$ ) *	1.00	Redundant
	1.05	Non-redundant
Importance ( $\eta_I$ ) **	0.90	Temporary Bridges
	0.95	ADT < 500
	1.00	500 ≤ ADT ≤ 40,000
	1.05	Major river crossing or ADT > 40,000 on bridge or Mainline interstate on bridge

\* Beam type superstructures with 4 or more beams per span are considered redundant

\*\* Use Importance load modifier for design of the superstructure only, except do not apply to deck designs for deck-on-girder type bridges. Use only on new bridges.

**3.3 Permanent Loads (Dead and Earth) [3.5]**

To reduce the number of load factors considered through the design process, use a value of 0.020 ksf for the future wearing surface load and combine with the other component dead loads (DC loads). Also, combine the load due to a concrete wearing course with other DC loads. Apply utility loads as DW loads with the appropriate AASHTO load factor.

Table 3.3.1 lists unit weights for a number of materials. Designers should note that several of these items differ slightly from the values contained in Section 3 of the LRFD Specifications.

**Table 3.3.1**  
**MnDOT Standard Unit Weights**

Material	Unit Weight (kcf)
Bituminous Wearing Course	0.150
Cast-In-Place Concrete	0.150
Precast Concrete	0.155
Precast Box Culvert	0.150
Compacted Fill on Box Culverts	0.120
Standard Fill	0.120
Steel	0.490
Timber	0.050
Water	0.0624

**3.4 Live Loads**  
**[3.6]**

HL-93 is the designation for the calibrated design live load provided in the LRFD Specifications. It should be considered the normal design load for MnDOT highway structures.

For pedestrian bridges, in addition to the pedestrian live load, design for a maintenance vehicle live load equivalent to an H-5 truck for deck widths from 6 to 10 feet, and an H-10 truck for wider decks. Use of the dynamic load allowance is not required with the maintenance vehicle.

Where appropriate, additional live loads should be considered. Additional live loads might include:

- MnDOT bridge inspection vehicle loads on bridges with large overhangs.
- MnDOT standard permit trucks on complex bridge types such as curved steel or post-tensioned concrete boxes. Discuss with the Bridge Ratings Engineer.
- Incorporate a live load surcharge into the design when construction or maintenance equipment will operate adjacent to retaining walls and abutments.

**3.4.1 HL-93 Live Load, LL**  
**[3.6.1.2]**

Use the design truck, fatigue truck, design tandem, truck train and lane loads described in the LRFD Specifications.

For simple spans, Tables 3.4.1.1 and 3.4.1.2 at the end of this section list the unfactored moments and shears for HL-93 loading on span lengths between 1 and 200 feet.

For continuous beam spans, internal studies have led to MnDOT modifications to the double truck live load given in LRFD Article 3.6.1.3.1. The modifications ensure adequate load ratings for the MnDOT standard permit trucks. In lieu of 90% of the HL-93 double truck stated in the LRFD Specifications, use the following live load for determining negative moments and interior pier reactions:

- For new bridges with longest span  $\leq 60$  feet, apply:  
 $1.25 \cdot [(\text{HL-93 double truck w/ dynamic load allowance}) + \text{lane load}]$
- For new bridges with longest span  $> 60$  feet, apply:  
 $1.10 \cdot [(\text{HL-93 double truck w/ dynamic load allowance}) + \text{lane load}]$
- For evaluation of existing bridge superstructures and substructures only, analyze per LRFD Article 3.6.1.3.1 instead of the MnDOT modifications above. Analyze for HL-93 live load and all the MnDOT G-80 and G-07 standard permit trucks. (Refer to Appendix F of the *MnDOT Bridge Load Rating and Evaluation Manual* for standard permit truck configurations.) For existing bridges that require repair, consider strengthening to a level of live load that meets the MnDOT modifications given above for a new bridge.
- Do not use the double tandem loading described in LRFD Article C3.6.1.3.1.

Note that these modifications apply to continuous beam spans only. For simple spans, follow LRFD Article 3.6.1.3.1 as written for determination of interior pier reactions.

**3.4.2 Multiple  
Presence Factor,  
MPF  
[3.6.1.1.2]**

When a structure is being evaluated for load cases involving more than two lanes of traffic a reduction factor or multiplier can be used. This factor recognizes the reduced probability that all lanes will be fully loaded at the same time. Note that the LRFD Specifications require a 1.2 factor to be used for the design of structures carrying a single lane of traffic.

**3.4.3 Dynamic  
Load Allowance, IM  
[3.6.2]**

What was known as impact in the Standard Specifications is called dynamic load allowance in the LRFD Specifications. The base dynamic load allowance factors are presented in LRFD Table 3.6.2.1-1. Designers should note that the base values are reduced for buried components and for wood structures.

**3.4.4 Pedestrian  
Live Load, PL  
[3.6.1.6]**

Pedestrian live loads vary with the function of the bridge. For conventional highway bridges with sidewalks wider than two feet, use an intensity of 0.075 ksf.

For pedestrian bridges, refer to the *Guide Specifications for Design of Pedestrian Bridges* for the pedestrian live load to be used.

**3.4.5 Braking  
Force, BR  
[3.6.4]  
[3.6.1.1.1]**

Use judgment when applying braking forces to a structure. For one-way bridges, apply the braking force in all AASHTO defined design lanes. For bridges striped as two-lane, two-way bridges, apply the braking force in one direction in both traffic lanes. For two-way bridges with more than two striped traffic lanes, determine the traffic direction with the greatest width (including width of any sidewalks and pedestrian trails adjacent to traffic) and apply the braking force to the number of AASHTO defined design lanes that fit within that width.

The dynamic load allowance factor is not applied to braking forces. However, multiple presence factors are to be used.

For pier design, braking forces are to be applied at a height 6 feet above the roadway surface and in a longitudinal direction. In bridges where there is not a moment connection between the superstructure and substructure (i.e., beam bridges on bearings), the braking force can be assumed to be applied to the pier at the bearings.

**3.4.6 Centrifugal  
Force, CE  
[3.6.3]**

Similar to braking forces, multiple presence factors are to be applied to the centrifugal force, while the dynamic load allowance is not applied.

Apply the centrifugal force at a height of 6 feet above the top of the deck.

**3.4.7 Live Load  
Application to  
Buried Structures**

For buried structures, a lane plus a design truck or tandem is applied to the roadway and distributed through the fill. If the fill is 2 feet or less, the live load is applied as a footprint to the top of the structure. For fills over 2 feet, the footprint load spreads out through the soil fill. Refer to Article 12.2.3 of this manual for more information on application of live load to box culverts.

**3.4.8 Live Load  
Surcharge, LS  
[3.11.6]**

Retaining walls and abutments typically need to be designed for load combinations with live load surcharge. The equivalent soil heights to be used for different heights of abutments and retaining walls are provided in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.

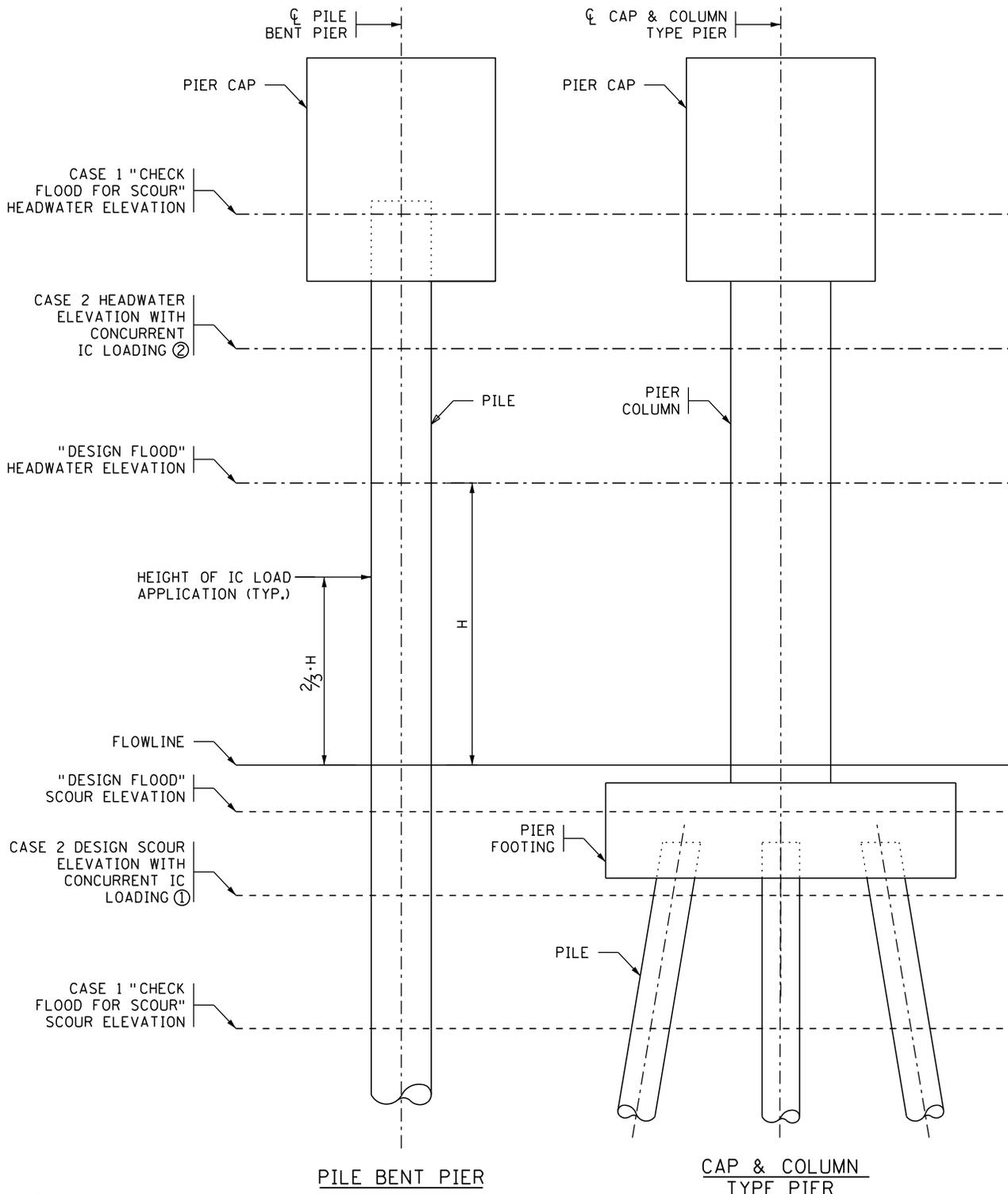
**3.5 Water Loads,  
WA  
[3.7]**

Some of the hydraulic event terminology used in the MnDOT hydraulic report differs from that used in the AASHTO LRFD Specifications (LRFD):

**[2.6.4.4.2 and  
3.7.5]**

- The “MnDOT design flood” for a structure is based on the average daily traffic that passes over the structure with the maximum design flood being a 50-year flood. (Refer to Section 3.2 of the MnDOT Drainage Manual for more information.) This is used as part of a roadway and surrounding property risk assessment done by the Hydraulics Section.
- The “LRFD design flood” for a structure is the lesser of the overtopping or 100-year flood. In other words, if overtopping occurs at a flood of less intensity than the 100-year flood, the “LRFD design flood” equals the overtopping flood. Otherwise, the “LRFD design flood” equals the 100-year flood. Use the “LRFD design flood” water and scour elevations (the 100-year flood is called out as the “basic flood” in the MnDOT hydraulic report) when analyzing piers for stream pressure loads under the strength and service limit states.
- The “check flood for scour”, as defined by LRFD, is the lesser of the overtopping or 500-year flood. In other words, if overtopping occurs at a flood of less intensity than the 500-year flood, the “check flood for scour” equals the overtopping flood. Otherwise, the “check flood for scour” equals the 500-year flood. Use the LRFD “check flood for scour” water and scour elevations to analyze piers for stream pressure (WA) loads under the extreme event limit state as follows (refer to Figure 3.5.1):
  - Case 1: Check piers using Extreme Event II for the full “check flood for scour” water and scour elevations. Do not include any BL, IC, CT, or CV loads for this check.
  - Case 2: Check piers using Extreme Event II for applicable BL, IC, CT, or CV loads.
    - For calculation of WA, use a headwater elevation equal to the average of the LRFD “design flood” and LRFD “check flood for scour” water elevations.
    - For the scour elevation, use the lower of:
      - The average of the flowline elevation and the LRFD “check flood for scour” elevation.
      - The LRFD “design flood” scour elevation.
    - Apply IC at the elevation given in BDM Article 3.8.

Design structural elements for both the no scour condition and the anticipated scour condition.



- ① THE LOWER OF:
  - AVERAGE OF FLOWLINE ELEVATION AND "CHECK FLOOD FOR SCOUR" ELEVATION
  - "DESIGN FLOOD" SCOUR ELEVATION
- ② AVERAGE OF "CHECK FLOOD FOR SCOUR" HEADWATER ELEVATION AND "DESIGN FLOOD" HEADWATER ELEVATION

Figure 3.5.1

**3.6 Wind Loads**  
**[3.8]**

Wind loads are based on the design 3-second gust wind speeds given in LRFD Table 3.8.1.1.2-1. Use a design 3-second gust wind speed of 115 mph for the Strength III limit state.

**3.6.1 Wind Load on Structure, WS**  
**[3.8.1.2 & 3.8.2]**

For design of substructures, use the following guidance regarding wind loads applied to ornamental metal railing or chain link fence:

- For Standard Figures 5-397.160 and .161, Ornamental Metal Railing with Fence (Design T-3), assume 50% of the combined rail/fence surface area is solid.
- For Standard Figures 5-397.162 and .163, Ornamental Metal Railing (Design T-4), assume 30% of the rail area is solid.
- Calculate the rail surface area for other standard and non-standard ornamental metal rails.
- For chain link fence, assume 30% of the fence area is solid.
- When determining the moment arm for pier design due to wind acting on the superstructure, assume the wind pressure acts on the full height of the ornamental metal rail or chain link fence.

Do not use these loads for ornamental metal railing or chain link fence design. Refer to LRFD Section 13 for railing design.

The vertical overturning wind load described in LRFD Article 3.8.2 must also be considered in design.

**3.6.2 Wind on Live Load, WL**  
**[3.8.1.3]**

Consider the force effects of wind on live load for the Strength V and the Service I load combinations.

Apply the wind on live load forces at a height 6 feet above the top of the deck. In bridges where there is not a moment connection between the superstructure and substructure (i.e., beam bridges on bearings), the longitudinal component of the wind on live load force can be assumed to be applied to the pier at the bearings.

**3.7 Earthquake Effects, EQ**  
**[3.10]**

All of Minnesota is in Seismic Zone 1 with acceleration coefficients varying between 2 and 3 percent. With very small acceleration coefficients, earthquake forces will rarely govern the design of MnDOT structures. However, Seismic Zone 1 structures must satisfy AASHTO requirements pertaining to the length of superstructure bearing seats and the horizontal design connection force between the superstructure and substructure.

**[4.7.4.4]**

For expansion bearings, check that the actual length of bearing seat,  $N_{act}$ , satisfies LRFD Article 4.7.4.4 using a Percentage  $N$  equal to 75.

**[3.10.9.2]**

For fixed bearings and anchors, MnDOT has modified the required horizontal connection force given in AASHTO. Design for a minimum horizontal connection force equal to 15% of the Strength I limit state vertical reaction.

**3.8 Ice Loads, IC  
[3.9]**

The design ice load is 1.5 feet of ice with a crushing strength of 32.0 ksf. Assume the ice load is applied at a height two-thirds of the distance from the flowline elevation to the lesser of the 100-year flood or overtopping flood high water elevation. Use a friction angle  $\theta_f$  equal to 0 degrees between the ice and pier nose.

**3.9 Earth Pressure,  
EV, EH or ES  
[3.5.1, 3.5.2]  
[3.11.5, 3.11.6]**

For cast-in-place cantilever concrete retaining walls, refer to the "Basis of Design" found on standard plan sheet 5-297.639 for determination of earth pressure loads. For other types of retaining walls, follow the current *AASHTO LRFD Bridge Design Specifications*.

For applications with level backfill other than retaining walls, simplified equivalent fluid methods can be used for determination of lateral earth pressure loads (EH). For parapet and semi-integral abutment stems, design for an active earth pressure of 0.033 kcf equivalent fluid weight. For level backfill applications where at-rest earth pressures cannot be relieved, design for an equivalent fluid weight of 0.060 kcf. Assume that the horizontal resultant for lateral earth pressures acts at a height of  $H/3$ .

For integral abutments and semi-integral abutment diaphragms, design for passive earth pressure loads. See Article 11.1.1 of this manual for load application.

For the vertical earth loads (EV) applied to pier footings, use a maximum load factor of 1.35 and a minimum load factor of 0.90.

**3.10 Temperature,  
Shrinkage, Creep,  
Settlement, TU, SH,  
CR, SE [3.12]**

Temperature, shrinkage, creep, and settlement produce several structural effects. They generate internal forces, redistribute internal forces, and produce movements.

As an alternative to AASHTO, the *CEB-FIP Model Code for Concrete Structures, 1990*, may be used to determine time dependent effects of concrete in post-tensioned structures.

**3.10.1**  
**Temperature**  
**Effects**

One of the most ambiguous tasks for bridge designers is the determination of the appropriate temperature range and corresponding deformations for use in calculating force effects on a structure. Past MnDOT practice has been to design concrete frames for a 45°F temperature fall and a 35°F temperature rise, a temperature range smaller than what the bridge will actually experience during its service life. This method dates back to the 1920s, and the reduced temperature range should be considered a “rule of thumb” that was applied to typical bridges using simplified analysis methods of the time. No notable performance issues have been attributed to application of a lower thermal temperature range when applied to pier frames or relatively short span bridges. On complicated, longer span bridge frames, longitudinal thermal effects become a larger issue that designers should not ignore. Therefore, the following policy is to be used for application of thermal loads on typical and non-typical bridges.

**Typical Bridges**

Typical bridges include routine multiple span prestressed beam, steel beam, and slab bridges that have the following:

- Two or fewer fixed piers.
- Piers less than 30 feet tall.
- Four spans or fewer, or that consist of units with four spans or fewer (where a unit is defined as the number of spans between expansion joints).
- A grade of 3% or less.

**[3.12.2.1]**

For typical bridges, use LRFD Procedure A for internal pier frame forces due to thermal expansion. For concrete frames, Procedure A allows for a temperature range of 80°F. Use a base construction temperature of 45°F, which corresponds to designing for thermal force effects due to a 45°F temperature fall and a 35°F temperature rise. In addition, apply the strength limit state load factor of 0.5 for calculation of thermal force effects and use gross section properties in the analysis. The 0.5 load factor accounts for the reduction in thermal forces due to cracking of the concrete.

**[3.12.2.2]**

For longitudinal effects, use a temperature range of 150°F (-30°F to 120°F), which is the approximate range given by LRFD Procedure B for Minnesota’s climate. Use a base construction temperature of 45°F and apply the strength limit state load factor of 0.5 for calculation of thermal force effects while using gross section properties in the analysis. Also, see Article 14.1 of this manual for guidance on fixity and thermal movements.

Design expansion joint openings for movements associated with a temperature range of 150°F (-30°F to 120°F). For strip seal expansion

joints, use a load factor for movement of 1.0. (Note that this value differs from the LRFD Specifications based on past performance of joints in Minnesota.) For modular expansion joints, use a load factor for movement of 1.2 per LRFD Article 3.4.1. See Article 14.2 of this manual for more guidance on expansion joints.

Design bearings for movements associated with a temperature range of 150°F (-30°F to 120°F) and a base construction temperature of 45°F. For computation of movement for the elastomeric pad minimum compressive stress check, use a load factor of 1.0. For computation of movement to determine minimum elastomer thickness, use a load factor of 1.3. (Note that these load factors differ from the LRFD Specifications and are based on past performance of elastomeric bearings in Minnesota.) For computation of movement for design of pot and disc bearings, use a load factor of 1.2.

### **Non-Typical Bridges**

Non-typical bridges are those with tall or slender piers or those with long spans. For these bridges, the pier stiffness is critical in determining movements and forces, and a refined analysis must be used to reduce force effects due to thermal movements and other loads.

#### **[3.12.2.2]**

For non-typical bridges, use a temperature range of 150°F (-30°F to 120°F) for longitudinal effects, which is the approximate range given by LRFD Procedure B for Minnesota's climate. When analyzing bridges with this larger thermal range, the designer must consider the following in the analysis:

- Pier stiffness – Use refined method to determine the appropriate percentage of gross stiffness along the height of the pier.
- Bearing fixity and flexibility – Account for the stiffness of expansion bearings in determination of the overall bridge movements.
- Construction method, staging, temperature range at erection, and its effect on the connectivity of the structural system.
- Foundation stiffness – Elastic shortening of the piles provides a significant relaxation to forces applied to the pier. Also, horizontal displacements of piling will provide moment reduction.
- For joint and bearing sizing, use a 150°F range at Service Limit State conditions. Use a thermal movement load factor of 1.2. Also use this movement to determine horizontal force requirements for guided bearings.
- For Strength Limit State, use a thermal load factor of 1.0 with the 150°F range for longitudinal force effects. For transverse effects within individual pier frames, an 80°F range with a 45°F base construction temperature may be used.

A 3-D model of the bridge with appropriate elastic restraints at supports may be required (especially for curved bridges) to determine the direction of movement, magnitude of thermal forces, and interaction between piers for determination of the appropriate cracked section reduction in stiffness. The final solution may require several iterations and may be bracketed using an upper-bound and lower-bound stiffness matrix (i.e., - gross sections, partially cracked sections, etc.) so that the final solution falls within an acceptable range for the particular structure.

In cases where several piers are fixed to the superstructure, consideration of ambient temperature at anticipated time of construction (including adjustments for closure pours as necessary) should be considered. Setting of bearings and joints within the structure may require special provisions that call for contractor submittals which state the intended method of bearing and joint installation to obtain a neutral position at the mean temperature.

Some non-typical bridges will consist of multiple units (where a unit is defined as the number of spans between expansion joints) with multiple bridge types, where not all units are non-typical. For example, a major river crossing may consist of 3 units: a multi-span slab type approach unit, a single main span tied arch unit, and a pretensioned concrete beam approach unit. If the approach units fit the typical bridge category, a refined analysis for pier stiffness determination is not required for the approach units. However, use of a thermal movement load factor of 1.2 is still required for joint and bearing sizing in the typical units.

### ***3.10.2 Shrinkage Effects***

Use a design relative humidity to 73% for concrete shrinkage computations.

### ***3.11 Pile Downdrag, DD***

For situations where long friction piles or end-bearing piles penetrate through a soft, compressible, top layer of material, long term settlement of the soft layer may introduce a downdrag load to the pile as it grips the pile through negative skin friction. An estimate of the downdrag load will be given in the Foundation Engineer's Memo and the amount of downdrag load to consider in design will be specified in the Foundation Recommendations. See Section 10.1.2 of this manual for more discussion on downdrag.

### ***3.12 Friction Forces, FR*** ***[3.13]***

Friction forces are used in the design of several structural components. For example, substructure units supporting bearings with sliding surfaces

should be designed to resist the friction force required to mobilize the bearing.

### **3.12.1 Sliding Bearings**

LRFD Table 14.7.2.5-1 provides design coefficients of friction for PTFE sliding surfaces.

### **3.12.2 Soil/Backwall Interface and Soil/Footing Interface**

Use LRFD Table 3.11.5.3-1 to obtain the coefficients of friction between the backwall/footing and soil. When cohesionless backfill is used behind a vertical or near vertical wall, the friction between the backwall and the backfill can be ignored.

When evaluating the sliding resistance between a concrete and soil interface, a coefficient of 0.80 shall be used. For cases where a shear key is utilized, the portion of the failure plane with soil on both sides should be evaluated with a coefficient of friction of 1.00.

### **3.13 Extreme Event**

The probability of extreme event loads occurring simultaneously is extremely small and therefore, is not to be applied concurrently. In some cases, extreme event loads are mutually exclusive. A vessel collision load can not occur when the waterway is iced over.

For the extreme event cases with ice (IC) or vessel collision (CV), evaluate bridges for 50% of the 500 year scour event depth.

#### **3.13.1 Vehicle Collision, CT [3.6.5]**

Designers need to be concerned with vehicle collision loads. Unprotected structural elements that may be struck bluntly by a vehicle or train shall be protected or be designed to resist the collision force. Review the Preliminary Plans to determine what is required. Also, see Section 11.2.3 of this manual for complete pier protection policy and requirements.

There are two documents which contain crash test criteria for bridge railings and barriers. They are *NCHRP Report 350* and the more recent *Manual for Assessing Safety Hardware*. The performance of barriers is classified with different test levels ranging from TL-1 to TL-6.

Decks supporting safety barriers designed to contain errant vehicles on bridges shall be designed for collision forces consistent with roadway standards. In most cases, the minimum standard for safety barriers on bridges carrying high speed traffic in Minnesota is Test Level 4 (TL-4). Under certain circumstances, reduced test level requirements may be

acceptable. For example, TL-3 may be adequate for buried structures. See Section 13 of this manual for additional guidance.

**3.13.2 Vessel Collision, CV [3.14]**

Structures within reaches of the Mississippi, Minnesota, and St. Croix rivers, and Lake Superior deemed navigable by the Corps of Engineers shall be designed to resist vessel collision loads.

**3.14 Uplift**

For curved bridges with skews or continuous bridges with spans that vary significantly, there is a possibility of uplift at the end supports. For situations where a sidespan is less than 70% of the adjacent continuous span, uplift should be considered. Uplift may occur during construction if deck placement is not sequenced properly or during service due to the application of live load if the spans are not balanced. If uplift occurs, the performance of the bearings and expansion joints may be compromised. When evaluating a structure for uplift the load factors for permanent load should be reviewed. Minimum and maximum factors shall be combined for different elements to generate the most conservative or largest uplift force effect.

**[Table 3.4.1-2]**

**3.15 Construction Loads**

The designer must consider construction loads during design. The diaphragm spacing and top flange dimensions in the positive moment region of the steel beam superstructures are based on the construction load stage. Specialty structures such as segmental concrete bridges have unique construction loads to consider during design that are explicitly defined. Unless project specific information is available or necessary, use the following loads:

**Formwork**

For conventional formwork (plywood, etc.) assume a uniform dead load of 0.010 ksf. In addition to dead loads, design concrete formwork for a construction live load of 0.050 ksf.

**Structural Elements**

Structural elements that support formwork are assumed to have a larger tributary area and consequently are to be designed for a smaller construction live load of 0.020 ksf.

Consider reconstruction loads when designing end diaphragms. At abutments, design end diaphragms to carry vertical jacking forces during bearing replacement.

**3.16 Deflections**  
**[2.5.2.6.2]**

MnDOT's maximum permitted live load deflection for highway bridges without sidewalks is  $L/800$ . For highway bridges with sidewalks, the limit is reduced to  $L/1000$ .

For typical deck-on-beam bridges that meet the LRFD Table 4.6.2.2.2b-1 and 4.6.2.2.2d-1 "Range of Applicability", use the following load distribution when computing deflections:

Live Load:

$$\text{Live Load Distribution Factor } LLDF_{\text{def}} = \text{MPF} \cdot \left( \frac{\text{Number of lanes}}{\text{Number of beams}} \right)$$

Dead Load:

$$\text{Dead Load (per beam) } DL_{\text{def}} = \left( \frac{\text{Total DC}}{\text{Number of beams}} \right)$$

For deck-on-beam bridges that fall outside the LRFD Table 4.6.2.2.2b-1 and 4.6.2.2.2d-1 "Range of Applicability", a 3D model may be used to determine deflections.

**Table 3.4.1.1**  
**Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions**  
**Simple Spans, One Lane, w/o Dynamic Load Allowance or Multiple Presence Factor**

Span (ft)	Moments				Shears and End Reactions		
	Truck (kip-ft)	Tandem (kip-ft)	Lane (kip-ft)	Span Pt. (%)	Truck (kip)	Tandem (kip)	Lane (kip)
1	8.0	6.3	0.1	0.50	32.0	25.0	0.3
2	16.0	12.5	0.3	0.50	32.0	25.0	0.6
3	24.0	18.8	0.7	0.50	32.0	25.0	1.0
4	32.0	25.0	1.3	0.50	32.0	25.0	1.3
5	40.0	31.3	2.0	0.50	32.0	30.0	1.6
6	48.0	37.5	2.9	0.50	32.0	33.3	1.9
7	56.0	43.8	3.9	0.50	32.0	35.7	2.2
8	64.0	50.0	5.1	0.50	32.0	37.5	2.6
9	72.0	62.5	6.5	0.50	32.0	38.9	2.9
10	80.0	75.0	8.0	0.50	32.0	40.0	3.2
11	84.5	92.0	9.3	0.40	32.0	40.9	3.5
12	92.2	104.0	11.1	0.40	32.0	41.7	3.8
13	103.0	115.9	13.4	0.45	32.0	52.3	4.2
14	110.9	128.3	15.5	0.45	32.0	52.9	4.5
15	118.8	140.6	17.8	0.45	34.1	43.3	4.8
16	126.7	153.0	20.3	0.45	36.0	43.8	5.1
17	134.6	165.4	22.9	0.45	37.6	44.1	5.4
18	142.6	177.8	25.7	0.45	39.1	44.4	5.8
19	150.5	190.1	28.6	0.45	40.4	44.7	6.1
20	158.4	202.5	31.7	0.45	41.6	45.0	6.4
21	166.3	214.9	34.9	0.45	42.7	45.2	6.7
22	174.2	227.3	38.3	0.45	43.6	45.5	7.0
23	182.2	239.6	41.9	0.45	44.5	45.7	7.4
24	190.1	252.0	45.6	0.45	45.3	45.8	7.7
25	198.0	264.4	49.5	0.45	46.1	46.0	8.0
26	210.2	276.8	53.5	0.45	46.8	46.2	8.3
27	226.1	289.1	57.7	0.45	47.4	46.3	8.6
28	241.9	301.5	62.1	0.45	48.0	46.4	9.0
29	257.8	313.9	66.6	0.45	48.8	46.6	9.3
30	273.6	326.3	71.3	0.45	49.6	46.7	9.6
31	289.4	338.6	76.1	0.45	50.3	46.8	9.9
32	307.0	351.0	81.1	0.45	51.0	46.9	10.2
33	324.9	363.4	86.2	0.45	51.6	47.0	10.6
34	332.0	375.0	92.5	0.50	52.2	47.1	10.9
35	350.0	387.5	98.0	0.50	52.8	47.1	11.2
36	368.0	400.0	103.7	0.50	53.3	47.2	11.5
37	386.0	412.5	109.5	0.50	53.8	47.3	11.8
38	404.0	425.0	115.5	0.50	54.3	47.4	12.2
39	422.0	437.5	121.7	0.50	54.8	47.4	12.5
40	440.0	450.0	128.0	0.50	55.2	47.5	12.8

**Table 3.4.1.2**  
**Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions**  
**Simple Spans, One Lane, w/o Dynamic Load Allowance or Multiple Presence Factor**

Span (ft)	Moments				Shears and End Reactions		
	Truck (kip-ft)	Tandem (kip-ft)	Lane (kip-ft)	Span Pt. (%)	Truck (kip)	Tandem (kip)	Lane (kip)
42	485.2	474.8	139.7	0.45	56.0	47.6	13.4
44	520.9	499.5	153.3	0.45	56.7	47.7	14.1
46	556.5	524.3	167.6	0.45	57.4	47.8	14.7
48	592.2	549.0	182.5	0.45	58.0	47.9	15.4
50	627.8	573.8	198.0	0.45	58.6	48.0	16.0
52	663.4	598.5	214.2	0.45	59.1	48.1	16.6
54	699.1	623.3	230.9	0.45	59.6	48.1	17.3
56	734.7	648.0	248.4	0.45	60.0	48.2	17.9
58	770.4	672.8	266.4	0.45	60.4	48.3	18.6
60	806.0	697.5	285.1	0.45	60.8	48.3	19.2
62	841.6	722.3	304.4	0.45	61.2	48.4	19.8
64	877.3	747.0	324.4	0.45	61.5	48.4	20.5
66	912.9	771.8	345.0	0.45	61.8	48.5	21.1
68	948.6	796.5	366.2	0.45	62.1	48.5	21.8
70	984.2	821.3	388.1	0.45	62.4	48.6	22.4
75	1070.0	887.5	450.0	0.50	63.0	48.7	24.0
80	1160.0	950.0	512.0	0.50	63.6	48.8	25.6
85	1250.0	1012.5	578.0	0.50	64.1	48.8	27.2
90	1340.0	1075.0	648.0	0.50	64.5	48.9	28.8
95	1430.0	1137.5	722.0	0.50	64.9	48.9	30.4
100	1520.0	1200.0	800.0	0.50	65.3	49.0	32.0
110	1700.0	1325.0	968.0	0.50	65.9	49.1	35.2
120	1880.0	1450.0	1152.0	0.50	66.4	49.2	38.4
130	2060.0	1575.0	1352.0	0.50	66.8	49.2	41.6
140	2240.0	1700.0	1568.0	0.50	67.2	49.3	44.8
150	2420.0	1825.0	1800.0	0.50	67.5	49.3	48.0
160	2600.0	1950.0	2048.0	0.50	67.8	49.4	51.2
170	2780.0	2075.0	2312.0	0.50	68.0	49.4	54.4
180	2960.0	2200.0	2592.0	0.50	68.3	49.4	57.6
190	3140.0	2325.0	2888.0	0.50	68.5	49.5	60.8
200	3320.0	2450.0	3200.0	0.50	68.6	49.5	64.0

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#### **4. STRUCTURAL ANALYSIS AND EVALUATION**

The analysis of bridges and structures is a mixture of science and engineering judgment. In most cases, use simple models with conservative assumptions to arrive at the design forces for various elements. For example, for straight beam bridges with small skews, use beam line models with approximate distribution factors to arrive at the design moments, shears, and reactions. For more complex structures or for situations where refinement offers significant benefits, a more refined analysis (e.g., grillage or 3-D) might be justified. Situations where this might be appropriate include curved bridges, bridges with large skews, or when evaluating the critical element of a bridge with marginal live load capacity. If the designer believes the bridge analysis requires a grillage model or that a complex bridge component requires a 3D model, the designer shall, in conjunction with the State Bridge Design Engineer, determine the appropriate level of analysis and modelling.

In all but the most complex bridges, time-dependent behavior will not be modeled. The impacts of creep, shrinkage, and relaxation will be accounted for by using code prescribed equations for these effects. While time-dependent material effects are not modeled, designers and evaluators of continuous post-tensioned structures should include secondary moments due to post-tensioning in their analysis.

Satisfying force equilibrium and identifying a load path to adequately transfer the loads to the foundations is the primary analysis goal for designers.

The remainder of this section contains guidance on a variety of topics. Topics include quality control and quality assurance, load distribution, load rating, substructure fixity, and lastly, LRFD usage.

##### **4.1 Design QC/QA Process**

Engineering software and spreadsheets play an important role in the design of bridges. The Bridge Office evaluates and utilizes vendor software and develops spreadsheets to assist office personnel. This process does not remove the responsibility of the designer to verify (through hand calculations, other programs, past experience, etc.) that results are accurate, cost efficient, constructible, and reasonable. The Bridge Design Automation Committee evaluates programs that may be used by in-house designers and maintains a list of approved spreadsheets.

##### **[4.4]**

As part of the quality control process, all components of the design, whether designed by hand or using computer programs, must be checked by a second engineer. Any discrepancies between the results of the

original design and the design check must be resolved as part of the quality control/quality assurance (QC/QA) process. Thorough checks for all designs are crucial. This goal is often more difficult to achieve when using vendor-supplied design products because of the user's inability to see the complete set of assumptions and computations within the software. Due to the varying intricacy of bridge elements, different levels of checking must be used.

### **Basic**

Basic components are primarily designed by hand calculations, by a spreadsheet, or with a vendor-supplied design application. Examples of bridge elements that may be reviewed using a basic level check include, but are not limited to, abutments, splices, bearings, and most cases of prestressed concrete beams.

A basic level check may be done in one of three ways:

- an independent set of calculations
- a line-by-line check of calculations
- using software that has been validated for a similar situation.

An independent set of calculations may be done by hand, spreadsheet, or using design software. To be considered a fully independent set of calculations, the second set cannot use the same software package or spreadsheet as the first. A comparison of input, intermediate output and final output values from the design and independent check calculation packages is also required.

If the design is performed using design software, the checking engineer must perform a complete assessment of all input values and a review of the output to confirm a reasonable answer. For a line-by-line check, every line of calculations must be verified by the checking engineer.

If an independent set of calculations is not completed, the checking engineer must handwrite initials on each page of calculations, computer input, and computer output that has been reviewed to indicate that the check has been performed. Preprinted checker initials are not acceptable as part of the quality control process. This applies both to line-by-line checks and designs performed using validated software.

Validation of software used to perform basic level checks may be accomplished through hand calculations or by replication of the results of the design examples given in this manual, where such an example exists. Verification of each step in the design process must be done. Once validation of the software has been completed, the process specified

under the basic check can be considered adequate. It is the designer's responsibility to verify that the validation that has been done remains current, i.e. that software changes are reviewed and current specification updates are included. The designer must include documentation in the calculation package stating that the software used has been validated.

### **Intermediate**

Intermediate components are those that are designed using a software design package, but whose outputs cannot easily be verified using hand computations and spreadsheets. Bridge elements requiring an intermediate level check include, but are not limited to, piers, straight steel girders, steel box girders, and prestressed beams that are flared or have variable width overhangs.

Unlike software packages that fall under the basic level check, validation of design software used for an intermediate level check is impossible because of the variety or complexity of the bridge component. Although running the design example from this manual, where one is available, provides some assurance in the software, there remain too many potential variables unchecked. Therefore, the software cannot be adequately validated, and an independent analysis is required for this type of analysis. A comparison of input, intermediate output and final output values from the design and independent check calculation packages is also required. The check may be performed by a second software package or via hand calculations or a spreadsheet. Depending on the complexity of the design, a hand check may use moderate simplifying assumptions. Sound engineering judgment must be used in making those assumptions. Input values that must be checked include geometry and live load distribution factors. At a minimum, output values must be compared for section properties, dead load moments and shears, live load moments and shears, and code checks. The checking engineer need not examine each load case generated by a program; however, load cases should be reviewed to validate loads were correctly combined and applied to find the maximum effects. Determination of critical live load cases for checking should be accomplished by load patterning.

### **Complex**

Complex bridge components are those that cannot reasonably be designed by hand or spreadsheet, even if moderate simplifications are made. Bridge elements that require a complex level check include, but are not limited to, concrete box girders, curved steel girders, and structures requiring a soil-structure interaction model.

The intricacies of these bridge elements require using two independent analyses with input and output compared at each stage of the design process. Verification of the results can only be completed using a second piece of software and comparing the modeling method, initial assumptions, and output results. A comparison of output values at each stage must be done, including, but not limited to, geometry, live load distribution, section properties, dead and live load moments and shears, and code checks.

## **4.2 Load Distribution**

### **[4.6.2]**

The LRFD Specifications encourage the use of either refined or approximate methods of analysis for determination of load distribution. The default analysis method for determination of the lateral load distribution for typical deck on beam bridges and slab span bridges is the approximate method of analysis given in the LRFD Specifications. Lateral live load distribution factors determined using the LRFD Specifications are dependent on multiple characteristics of each bridge and there are specific ranges of applicability for their use. Extending the application of such approximate methods beyond the limits requires sound and reasonable judgement. Otherwise refined analytical methods should be used.

### **4.2.1 Dead Load Distribution**

#### **Deck, Wearing Course, Future Wearing Surface, Railing, Barriers, and Medians**

For beam bridges, the dead load of the deck is distributed to the beams based on their respective tributary widths. Superimposed dead loads (wearing course, future wearing surface, railings, barriers, and medians), with the exception of sidewalk loads, are to be distributed equally to all beam lines.

For concrete slab bridges (reinforced or post-tensioned) the weight of the barrier loads should be distributed to the edge strip. For design of the interior strip, the weight of the barriers should be distributed across the entire width of the slab and combined with other superimposed dead loads.

#### **Sidewalks**

Distribute sidewalk loads to the beams by simple distribution except when checking load case 2 as specified in Article 4.2.3 of this manual.

#### **Miscellaneous Loads – Conduits, Sign Structure, etc.**

Conduit loads supported by hangers attached to the deck should be distributed equally to all beams. Sign structures, architectural treatment

panels, and sound walls, whose load acts entirely outside the exterior beam, should be assumed to be carried by the exterior beam.

#### **4.2.2 Live Load Distribution**

Equations and tables for live load distribution factors are provided in the LRFD Specifications.

##### **4.2.2.1 Steel and Prestressed Concrete Beams**

###### **[4.6.2.2]**

For typical beam bridges, use the live load distribution factor (LLDF) formulas provided in the LRFD Specifications for interior beam flexure (single lane, multiple lanes, and fatigue), and interior beam shear (single lane, multiple lanes, and fatigue). For exterior beams, use the lever rule and LLDF formulas to determine the amount of live load carried by the exterior beam. In addition, use the rigid cross section equation (LRFD C4.6.2.2.2d-1) for steel beam bridges. The number of diaphragms/cross frames found in steel beam bridges makes rigid cross-section rotation and deflection a valid behavior to consider. Use of the rigid cross section equation is not required for design of precast prestressed concrete exterior beams.

Unlike the Standard Specifications, the LRFD live load distribution factors (LLDF) for beam bridges are dependent on the stiffness of the components that make up the cross section [LRFD Equation 4.6.2.2.1-1]. Theoretically, the distribution factor changes for each change in cross section (at flange plate changes in plate girders, for example). However, this is more refinement than is necessary. For simple span structures a single LLDF (computed at midspan) may be used. For continuous structures, a single LLDF may be used for each positive moment region and for each negative moment region, with the moment regions defined by the dead load contraflexure points. For bridges with consistent geometry (same number of beam lines in each span, etc.) the largest positive moment LLDF may be used for all positive moment locations. Similarly, the largest negative moment LLDF may be used for all negative moment regions. Also note that for continuous structures, use the span length "L" as defined by LRFD Table 4.6.2.2.1-2 for LLDF calculations. ||

For skewed superstructures:

###### **[4.6.2.2.2e]**

- Apply the live load distribution reduction factor for moment per LRFD Article 4.6.2.2.2e.

###### **[4.6.2.2.3c]**

- Apply the live load distribution correction factor for shear to all beams and throughout the entire beam length.

**4.2.2.2 Slab Spans  
and Timber Decks  
[4.6.2.3]**

Design concrete slabs and timber decks using a one foot wide longitudinal strip. The LRFD Specifications provide equations for live load distribution factors (LLDF) that result in equivalent strip widths,  $E$ , that are assumed to carry one lane of traffic. Convert the equivalent strip width to a live load distribution factor for the unit strip by taking the reciprocal of the width.

$$\text{LLDF} = \frac{1}{E}$$

**4.2.3 Sidewalk  
Pedestrian Live  
Load  
[3.6.1.6]**

Unlike the Standard Specifications, no reduction in sidewalk pedestrian live load intensity based on span length and sidewalk width is provided in the LRFD Specifications.

- 1) Consider two loading cases when designing a beam bridge with a sidewalk: Use a pedestrian live load on the sidewalk equal to 0.075 ksf, and apply it in conjunction with a vehicular live load in the traffic lanes adjacent to the sidewalk. Use the lever rule to determine distribution of sidewalk dead load, pedestrian live load, and vehicular live load to outer beams.
- 2) Place vehicular live load on the sidewalk and in adjacent traffic lanes with no pedestrian live load on the sidewalk. For this load case, assume dead load, including sidewalk, is carried equally by all beams.

**4.3 Load Rating**

The bridge load rating determines the safe load carrying capacity. Ratings are calculated for a new bridge and are recalculated throughout the bridge's life as changes occur.

Unlike design, where only one benchmark or level of safety is used, two different levels have historically been used for load rating. These rating levels are referred to as the "inventory rating" and "operating rating". The inventory rating corresponds to the factors of safety or levels of reliability associated with new bridge designs. The operating rating corresponds to slightly relaxed safety factors or reliability indices and is used for infrequent, regulated loads. Calculations for overload permit evaluations and for bridge weight postings are made at the operating level.

The Design Data block on the front sheet of a set of bridge plans should contain the LRFR HL-93 operating rating factor for the bridge.

When the bridge plan is to the point where all the essential information for the superstructure is shown, the plan should be sent to the Bridge Rating Unit. They will calculate the operating rating for the bridge.

Bridges designed for the local road system are generally prepared by the local agency and/or their consultants. It is the responsibility of the local agency to assure that ratings are calculated and reported to the Bridge Asset Data Management Unit.

Detailed information on load rating of bridges in Minnesota can be found in Section 15 of this manual.

#### **4.4 Substructure Fixity**

The overall fixity of the bridge should be examined in detail for bridges on steep grades, moderate to severe curvature, or when the columns are tall or slender. The following guidelines for providing fixity at bearings should be followed.

For short bridges on steep grades, the down hill abutment should be fixed. For longer bridges the flexibility of each pier and its bearings need to be considered to determine the appropriate substructure units to fix.

If pier flexibility and geometry permit, a minimum of two fixed piers per expansion unit should be used. For very flexible piers, such as pile bents or slender columns, the expansion bearings may be redundant (the pier may move before the bearings begin to slide).

For typical prestressed I-beam bridges with two sets of bearings on each pier (per beam line), sufficient anchorage to the pier is provided by using one line of bearings with anchor rods at a fixed pier. For river piers and for spans over 145 feet, designers should fix both sets of bearings.

See Section 14 of this manual for additional guidance.

#### **4.5 Structural Models**

For redundant structures, the distribution of internal forces is dependent on member stiffnesses. Engineering judgement needs to be exercised when assigning member properties and boundary conditions to determine the internal forces of members.

Often a simplified method can be used to arrive at a solution. For example, instead of setting up a continuous beam model, design moments in pile bent pier caps can be determined in the following manner: Positive moment requirements can be determined by assuming

simple spans between the supporting piles. The required negative moment capacity can be computed assuming a propped cantilever for the outside spans and fixed/fixed boundary conditions for the interior spans.

#### **4.6 Design Methodology & Governing Specifications**

The *AASHTO LRFD Bridge Design Specifications* are extensive, but do not cover all bridge types. In addition, they were not written for bridge rehabilitation projects. MnDOT policy regarding these topics is given below.

##### **4.6.1 Pedestrian Bridges**

Design pedestrian bridges in accordance with the *LRFD Guide Specifications for Design of Pedestrian Bridges*. The pedestrian live load specified in the *AASHTO LRFD Bridge Design Specifications* is only for vehicular bridges that carry pedestrian traffic. The pedestrian bridge guide specifications address the design of pedestrian bridges.

##### **4.6.2 Repair Projects**

When repairing existing bridges, it is often not economically feasible to design the repaired structure to meet all current design code requirements, including live load capacity. To help establish uniform procedures for use on bridge repair projects, MnDOT developed the *Bridge Preservation and Improvement Guidelines* (BPIG). These guidelines are updated at regular intervals and provide a systematic approach to planning and performing bridge preservation and rehabilitation projects. The BPIG also includes condition and cost criteria for bridge replacement projects, as well as policies for upgrading substandard features like barriers and end posts. Appropriate bridge design standards have been established based on investment level, along with expected outcomes in terms of slowed deterioration, improved condition, or service life extension.

Bridge repair projects include all major bridge preservation and rehabilitation projects, which are defined as:

- Major bridge preservation: These projects involve extensive bridge repairs intended to extend the service life of structures while maintaining their existing design features. Some examples include joint replacements, deck patching and overlays, barrier replacements, and bridge painting projects.
- Bridge rehabilitation: These projects involve repairing deficiencies in structures and improving their geometrics and/or load-carrying capacity. Some examples include bridge widenings, deck replacements, and superstructure replacements.

The bridge designer will receive a copy of the *Bridge Repair Recommendations*, approved by the District, for each bridge in a proposed repair project. The MnDOT Regional Bridge Construction Engineer prepares the recommendations in accordance with the BPIG and specifies the scope of the bridge repair project.

Most repair projects were originally designed in accordance with the *AASHTO Standard Specifications for Highway Bridges*. Therefore, it may seem logical to design the repair using the same governing specifications. However, the *AASHTO Standard Specifications for Highway Bridges* is no longer being maintained, has not been updated since 2002, and has several documented deficiencies. Thus, it is appropriate for repair projects to be evaluated and designed using the current edition of the *AASHTO LRFD Bridge Design Specifications (LRFD)* along with the latest load and resistance factor rating (LRFR) requirements from the *Manual for Bridge Evaluation (MBE)*. The LRFD specifications are based on the latest research, incorporating the variability in material properties and loading, as well as being statistically calibrated to provide uniform reliability.

Therefore, the following applies to all bridge repair projects, regardless of original design code:

- Load rating evaluations for repair projects shall be done using LRFR procedures. These evaluations should be performed during the scoping phase of the project. For typical projects, the Bridge Ratings Engineer will develop the evaluation. For special structures, the Bridge Ratings Engineer and State Bridge Design Engineer will determine if assistance is required to complete the evaluation and who will perform it.
- For bridge rehabilitation projects, such as deck replacements, widenings, and superstructure replacements, design and analysis shall be done using LRFD procedures. Because these types of projects are a major investment and significantly extend service life, it is important to evaluate the bridges using current standards.
- For major bridge preservation projects that significantly increase dead load, like those with bridge rail modifications or those that increase the deck thickness, design and analysis shall be done using LRFD procedures.
- Major bridge preservation projects such as deck repairs, painting, mill and overlays, and joint replacements typically do not require

any analysis as part of the final plan development. However, these projects should include an up-to-date LRFR evaluation during the scoping phase of the project to assess potential areas of concern that may need to be addressed in the repair plan.

Minimum LRFR requirements for superstructures of bridge repair projects:

- As previously allowed in the BPIG, which required a minimum load factor rating of HS18 (0.9 x HS20 design vehicle), an LRFR inventory rating factor of 0.9 is the minimum acceptable level for the superstructure. This reduced inventory rating factor is considered acceptable recognizing that some of the service life of the bridge has transpired.
- For bridges with sidewalks, consider both of the load cases given in Article 4.2.3 of this manual. Consideration may be given to waiving Load Case 2 (vehicular load applied to the sidewalk) when the anticipated remaining life of the bridge is less than 10 years.

Minimum LRFR requirements for substructures (Note that this does not apply to foundations):

- Substructures are typically load rated only when significant additional loads will be applied. Evaluations may also be required if safety inspections note substantial deterioration or there is damage that indicates an inadequate design. Members that require evaluation will be noted in the repair recommendations.
- Traditional beam theory or strut-and-tie are both acceptable analysis methods for pier caps, provided the boundary conditions in AASHTO are met for the chosen analysis method.
- For bridge rehabilitation projects, the minimum acceptable LRFR inventory rating factor is 1.0 for substructures. (Because of rating software limitations regarding substructures, the minimum load rating requirement was set higher than for superstructures.)
- For major bridge preservation projects:
  - When the bridge currently has permit restrictions, the substructure inventory rating must be greater than or equal to the superstructure inventory rating.
  - When the bridge does not have current permit restrictions, the substructure inventory rating must be greater than or equal to 1.0, but need not exceed the superstructure inventory rating.

- The skin reinforcement requirements of AASHTO LRFD Article 5.7.3.4 need not be met for pier caps.
- For bridges with sidewalks, consider both of the load cases given in Article 4.2.3 of this manual. Consideration may be given to waiving Load Case 2 (vehicular load applied to the sidewalk) when the anticipated remaining life of the bridge is less than 10 years.

For cases where the required minimum inventory rating factor cannot be achieved, other options within the LRFR provisions of the MBE specifications and MnDOT policy can be considered. These options would need to be discussed on a case-by-case basis with the Bridge Ratings Engineer, Final Design Unit Leader, Bridge Construction Regional Engineer, State Bridge Design Engineer, State Bridge Construction and Maintenance Engineer, and the appropriate District personnel. In addition, a design exception can be recommended to the District based on investment level, cost, expected bridge service life, and service interruption risk.

#### **4.6.3 Railroad Bridges and Bridges or Structures near Railroads**

Railroad bridges are to be designed in accordance with the most current *AREMA Manual for Railway Engineering*.

Designers should be aware that oftentimes railroads have specific criteria for structural design of items carrying their tracks or in the vicinity of their tracks. The criteria vary from railroad to railroad. For example, the Duluth Mesabe & Iron Range Railway has a special live load. Other railroads have specific loading criteria and geometric limits for excavations near their tracks.

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## **5. CONCRETE STRUCTURES**

Reinforced and prestressed concrete are used extensively in bridge projects. In addition to general design guidance and information on detailing practices, this section contains three design examples: a three-span reinforced concrete slab superstructure, a 63 inch pretensioned I-beam, and a three-span post-tensioned concrete slab superstructure.

### **5.1 Materials**

For most projects, conventional materials should be specified. Standard materials are described in two locations: *MnDOT Standard Specifications for Construction* (MnDOT Spec.) and *Bridge Special Provisions*.

If multiple types of concrete or reinforcement are to be used in a project, it is the designer's responsibility to clearly show on the plans the amount of each material to be provided and where it is to be placed.

#### **5.1.1 Concrete**

MnDOT Spec. 2461 identifies and describes concrete mix types. Based on their strength, location of application, and durability properties, different mixes are used for various structural concrete components. Table 5.1.1.1 identifies the standard MnDOT concrete mix types to be used for different bridge components.

The four or five characters used to identify a concrete mix provide information on the properties of the mix. The first character designates the type of concrete (with or without air entrainment requirements). The second character identifies the grade of concrete. Each letter is associated with a different cement-void ratio. The third character in the label is the upper limit for the slump in inches. The fourth character identifies the coarse aggregate gradation. The fifth character, if present, identifies the type of coarse aggregate to be used. Note that there are two exceptions to the above: job mixes (JM) for box girders, and high performance concrete (HPC) mixes for bridge decks and slabs.

For HPC mixes, the first and second characters follow the description above. For monolithically poured decks, these are followed by either "HPC-M" or "LCHPC-M" (where the LC designates low cement). For decks that will receive a separate wearing course, these are followed by either "HPC-S" or "LCHPC-S" (where the LC designates low cement). For job mixes, the first character designates the type of concrete as above, but is followed by "JM" for mixes that will be determined by the Contractor.

In general, the standard concrete design strength is 4 ksi, and air entrained concretes are to be used for components located above footings and pile caps to enhance durability.

**Table 5.1.1.1 Design Concrete Mix Summary**

Location/Element	MnDOT Concrete Mix Designation	Design Compressive Strength (ksi)	Maximum Aggregate Size (in)
Cofferdam seals	1X62	5.0	1
Cast-in-place concrete piles and spread footing leveling pads	1P62	3.0	2
Drilled shafts	1X62 3X62	5.0 5.0	1 1
Footings and pile caps	1G52	4.0	1 ½ *
Abutment stems, wingwalls, cast-in-place wall stems, pier columns, and pier caps	3B52	4.0	1 ½ *
Integral abutment diaphragms and pier continuity diaphragms	Same mix as used in deck	4.0	1
Pretensioned superstructures	1W82 or 3W82	5.0 – 9.0 at final 4.5 – 7.5 at initial	1
Cast-in-place and precast box girders	3JM	6.0 or higher	1
Monolithic decks and slabs	3YHPC-M, 3YLCHPC-M or 3Y42-M	4.0	1
Decks and slabs that will receive a 2 inch concrete wearing course	3YHPC-S, 3YLCHPC-S or 3Y42-S	4.0	1
Barriers, parapets, medians, and sidewalks	3S52	4.0	1
Concrete wearing course	3U17A	4.0	5/8
MSE wall panels, PMBW blocks, and noisewall panels	3Y82	4.0	1
Precast box culverts, arches, and 3-sided structures	3W82	5.0 or higher	1*

\* For determination of  $s_{xe}$  per LRFD 5.8.3.4.2, use max aggregate size  $a_g = 3/4''$

### Reinforced Concrete Sections

Base concrete modulus of elasticity computations on a unit weight of 0.145 kcf. Use a unit weight of 0.150 kcf for dead load calculations.

For structural modeling (determining design forces and deflections), use gross section properties or effective section properties. For redundant structures with redundant and nonprismatic members, model with nonprismatic elements.

#### [5.4.2.4-1]

For reinforced concrete elements, use:  $E_c = 33,000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$

For checks based on strength (design of reinforcement, maximum reinforcement), use conventional strength methods (reinforcement yielding, Whitney equivalent stress block, etc.).

For checks based on service loads (fatigue, crack control, etc.), use cracked sections with reinforcing steel transformed to an equivalent amount of concrete.

**Prestressed Concrete Elements**

When computing section properties, use a modular ratio of 1 for the prestressing strands.

For pretensioned beams (M, MN, MW, and RB) fabricated with high-strength concrete (greater than 6.0 ksi), compute the modulus of elasticity with the ACI 363 equation below:

$$E_c = 1265 \cdot \sqrt{f'_c} + 1000 \quad (\text{where } f'_c \text{ and } E_c \text{ are in ksi})$$

For all other pretensioned and post-tensioned elements, compute the modulus of elasticity using AASHTO LRFD Equation 5.4.2.4-1, with  $K_1 = 1$  and  $w_c = 0.150$  kcf.

For both pretensioned and post-tensioned elements, use a unit weight of 0.155 kcf for dead load calculations.

Table 5.1.1.2 summarizes concrete properties for analysis and design:

**Table 5.1.1.2  
Concrete Properties**

Parameter	Equation/Value
Unit Weight	Reinforced Concrete Elements: $w_c = 0.145$ kcf for calculation of $E_c$ $w_c = 0.150$ kcf for dead load calculation Pretensioned and Post-tensioned Elements: $w_c = 0.150$ kcf for calc. of $E_c$ (except pretensioned beams) $w_c = 0.155$ kcf for dead load calculation
Modulus of Elasticity	Pretensioned Beams: $E_c$ (ksi) = $33,000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$ where $f'_c \leq 6$ ksi $E_c$ (ksi) = $1265 \cdot \sqrt{f'_c} + 1000$ where $f'_c > 6$ ksi All Other Concrete Elements: $E_c$ (ksi) = $33,000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f'_c}$
Thermal Coefficient	$\alpha_c = 6.0 \times 10^{-6} = \text{in/in/}^\circ\text{F}$
Shrinkage Strain	Reinf. Conc.: $\epsilon_{sh} = 0.0002$ @ 28 days and $0.0005$ @ 1 year Prestressed Concrete: per LRFD Art. 5.4.2.3
Poisson's ratio	$\nu = 0.2$

### 5.1.2 Reinforcing Steel

Reinforcing bars shall satisfy MnDOT Spec 3301. ASTM A615 Grade 60 deformed bars (black or epoxy coated) should be used in most circumstances. In some cases, Grade 75 stainless steel bars will be required in the bridge deck and barrier (see Tech. Memo No. 11-15-B-06 *Policy on the Use of Stainless Steel Reinforcement in Bridge Decks & Barriers*). Use  $f_y = 75$  ksi when designing with stainless steel bars. Always use stainless steel (either Grade 60 or 75 is adequate for this situation) for the connecting bar between approach panel and end diaphragm at integral and semi-integral abutments.

In specialized situations and with the approval of the State Bridge Design Engineer, welding to reinforcement may be used. ASTM A706 Grade 60 bars must be used for applications involving welding.

The modulus of elasticity for mild steel reinforcing ( $E_s$ ) is 29,000 ksi.

All reinforcement bars, except stainless steel bars and bars that are entirely embedded in footings, shall be epoxy coated.

### 5.1.3 Reinforcement Bar Couplers

Contractors select reinforcement bar couplers that meet the requirements stated in MnDOT Spec. 2472.3.D.2. In general, the couplers need to:

- Provide a capacity that is 125% of the nominal bar capacity.
- Be epoxy coated.
- Satisfy fatigue testing requirements of NCHRP Project 10-35 (12 ksi).

### 5.1.4 Prestressing Steel

Uncoated low-relaxation 7-wire strand or uncoated deformed, high-strength bars are acceptable prestressing steels. Strands shall conform to ASTM A416. Bars shall conform to ASTM A722.

Use the following properties for prestressing steel:

Tensile strength:  $f_{pu} = 270$  ksi for strands

$f_{pu} = 250$  ksi for bars

Yield strength:  $f_{py} = 243$  ksi for strands

$f_{py} = 120$  ksi for bars

Elastic Modulus:  $E_p = 28,500$  ksi for strands

$E_p = 30,000$  ksi for bars

Standard 7-wire prestressing strand area,  $A_{ps}$ :

$\frac{3}{8}$ " diameter strand: 0.085 in<sup>2</sup>/strand

$\frac{1}{2}$ " diameter strand: 0.153 in<sup>2</sup>/strand

0.6" diameter strand: 0.217 in<sup>2</sup>/strand

### **5.1.5 Post-tensioning Hardware**

For post-tensioned concrete bridges, open ducts must be used for tendon passageways through the superstructure. Longitudinal ducts are typically 3 to 4 inches in diameter and must be sufficiently rigid to withstand the loads imposed upon them. The preferred material for longitudinal ducts is corrugated plastic (HDPE). Transverse ducts are typically smaller, containing from 1 to 4 strands. Because the transverse ducts are relatively close to the top of the deck with heavy applications of corrosive de-icing chemicals, corrugated plastic ducts are required. The anchor head is typically galvanized or epoxy coated based on project needs. Discuss the protection requirements with the State Bridge Design Engineer.

Tendon anchorage devices are required at the ends of each duct. Anchorages should be shown and indicated on the drawings. Detailing is unnecessary because the post-tensioning supplier will provide these details in the shop drawings for the post-tensioning system. Designers must consider the local zone anchorage reinforcement (typically spiral reinforcement) provided by potential suppliers to allow adequate room for the general zone reinforcement designed and detailed in the bridge plans.

### **5.2 Reinforcement Details**

Practices for detailing a variety of reinforced concrete elements are presented in this section. These include standard concrete cover and bar spacing dimensions, plus a variety of specific design and detailing instructions.

Reinforcing details are intended to provide a durable structure with straightforward details. Details must be constructible, allowing steel to be placed without undue effort, and provide adequate clear cover and adequate space between reinforcement to permit the placement of concrete.

#### **5.2.1 Minimum Clear Cover and Clear Spacing**

The minimum clear cover dimension to reinforcement varies with the location in the bridge. It varies with how the component is constructed (precast, cast in forms, cast against earth) and the exposure the element has to de-icing salts. In general, minimum covers increase as control over concrete placement decreases and as the anticipated exposure to de-icing salts increases. Following is a list of structural components and the corresponding minimum clear cover. For components that are not listed, a 2" minimum clear cover is required unless it is shown differently in the Bridge Office standards.

## Foundations

### *Top Bars*

- Minimum clear cover is 3 inches.

### *Bottom Bars, Spread Footing*

- Minimum clear cover to the bottom concrete surface is 5 inches.
- Minimum clear cover to the side concrete surface is 3 inches.

### *Bottom Bars, Pile Cap w/ Pile Embedded 1 foot*

- Rest directly on top of trimmed pile.

### *Bottom Bars, Pile Cap Alone or Where Pile Cap is Cast Against a Concrete Seal, w/ Pile Embedded More Than 1 foot*

- Minimum clear cover is 3 inches to bottom of pile cap.

## Abutments and Piers

- Standard minimum clear cover for all bars is 2 inches (vertical and horizontal).
- At rustications, the minimum horizontal clear cover varies with the size of the recess. For recesses less than or equal to 1 inch in depth and less than or equal to 1 inch in width, the minimum clear cover is 1.5 inches. For all other cases, the minimum clear cover is 2 inches.
- Minimum clear distance between reinforcement and anchor rods is 2 inches.
- In large river piers with #11 bars or larger that require rebar couplers, minimum clear cover to bars is 2.5 inches.

## Decks and Slabs

### *Top Bars, Roadway Bridge Deck or Slab*

- Minimum clear cover to the top concrete surface is 3 inches.
- Minimum horizontal clear cover is 2 inches.

### *Top Bars, Pedestrian Bridge Deck*

- Minimum clear cover to the top concrete surface is 2 inches.

### *Bottom Bars, Deck*

- Minimum clear cover to the bottom concrete surface is 1 inch.
- Minimum horizontal clear cover from the end of the bar to the face of the concrete element is 4 inches.
- Minimum horizontal clear cover from the side of a bar to the face of the concrete element is 2 inches.

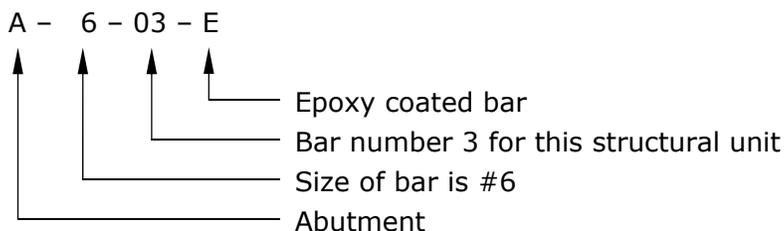
### *Bottom Bars, Slab*

- Minimum clear cover to the bottom concrete surface is 1.5 inches.
- Minimum horizontal clear cover from the end of the bar to the face of the concrete element is 4 inches.
- Minimum horizontal clear cover from the side of a bar to the face of the concrete element is 2 inches.

**5.2.2 Reinforcing Bar Lists**

For numbering of reinforcing bars, the first character is a unique alpha character for the given structural element. The first one or two digits of the bar mark indicate the U.S. Customary bar size. The last two digits are the bar's unique sequential number in the bar list for that substructure or superstructure unit. A suffix "E" indicates the bar is epoxy coated, "G" indicates the bar is galvanized, "S" indicates the bar is stainless steel, "Y" indicates a Grade 75 epoxy coated bar, and "Z" indicates a Grade 75 plain bar.

For example, an A603E bar could be decoded as follows:



The cross-sectional areas, diameters, and weights of standard reinforcing bars are provided in Table 5.2.2.1.

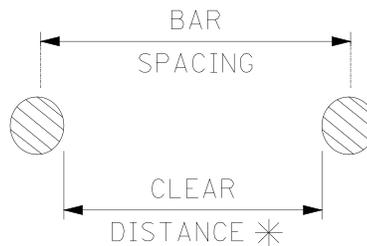
**Table 5.2.2.1  
Reinforcing Steel Sizes and Properties**

U.S. Customary Bar Size	Area of Bar (in <sup>2</sup> )	Diameter of Bar (in)	Weight of Bar (lb/ft)
#3	0.11	0.375	0.376
#4	0.20	0.500	0.668
#5	0.31	0.625	1.043
#6	0.44	0.750	1.502
#7	0.60	0.875	2.044
#8	0.79	1.000	2.670
#9	1.00	1.128	3.400
#10	1.27	1.270	4.303
#11	1.56	1.410	5.313
#14	2.25	1.693	7.650
#18	4.00	2.257	13.60

Table 5.2.2.2 lists the reinforcing steel area provided (per foot) for different sized bars with different center to center bar spacings.

**Table 5.2.2.2**  
**Average Area per Foot Width Provided by Various Bar Spacings (in<sup>2</sup> /ft)**

Bar Size Number	Nominal Diameter (in)	Spacing of Bars in Inches												
		3	3.5	4	4.5	5	5.5	6	7	8	9	10	11	12
3	0.375	0.44	0.38	0.33	0.29	0.26	0.24	0.22	0.19	0.17	0.15	0.13	0.12	0.11
4	0.500	0.80	0.69	0.60	0.53	0.48	0.44	0.40	0.34	0.30	0.27	0.24	0.22	0.20
5	0.625	1.24	1.06	0.93	0.83	0.74	0.68	0.62	0.53	0.47	0.41	0.37	0.34	0.31
6	0.750	1.76	1.51	1.32	1.17	1.06	0.96	0.88	0.75	0.66	0.59	0.53	0.48	0.44
7	0.875	2.40	2.06	1.80	1.60	1.44	1.31	1.20	1.03	0.90	0.80	0.72	0.65	0.60
8	1.000	3.16	2.71	2.37	2.11	1.90	1.72	1.58	1.35	1.19	1.05	0.95	0.86	0.79
9	1.128	4.00	3.43	3.00	2.67	2.40	2.18	2.00	1.71	1.50	1.33	1.20	1.09	1.00
10	1.270	---	4.35	3.81	3.39	3.05	2.77	2.54	2.18	1.91	1.69	1.52	1.39	1.27
11	1.410	---	---	4.68	4.16	3.74	3.40	3.12	2.67	2.34	2.08	1.87	1.70	1.56



\* Per LRFD 5.10.3.1.1, the minimum clear distance between bars in a layer shall be the greatest of:

- 1) 1.5 times the nominal diameter of the bar
- 2) 1.5 times the maximum size of the coarse aggregate \*\*
- 3) 1.5 inches

\*\* Per the current edition of *MnDOT Standard Specifications for Construction*

The weight of spiral reinforcement on a per foot basis is provided in Table 5.2.2.3. The standard spiral reinforcement is  $\frac{1}{2}$  inch diameter with a 3 inch pitch. When selecting the size of round columns, use outside dimensions that are consistent with cover requirements and standard spiral outside diameters.

Figure 5.2.2.1 through 5.2.2.5 contain development length (Class A lap) and tension lap splice design tables for epoxy coated, plain uncoated, and stainless steel reinforcement bars. Knowing the bar size, location, concrete cover, bar spacing, and class of splice, designers can readily find the appropriate lap length. The tables are based on 4 ksi concrete.

Figure 5.2.2.6 contains development length tables for bars with standard hooks. Values are provided for epoxy coated, plain uncoated, and stainless steel reinforcement bars. Standard hook dimensions are also included.

Figure 5.2.2.7 contains graphics that illustrate acceptable methods for anchoring or lapping stirrup reinforcement. Open stirrups must have the "open" end anchored in the compression side of the member. This anchorage consists of development of the bar or hook prior to reaching a depth of  $d/2$  or placing the hooks around longitudinal reinforcement. Detail closed double stirrups with a Class B lap. Also included in Figure 5.2.2.7 are stirrup and tie hook dimensions and a table showing minimum horizontal bar spacings for various concrete mixes.

**Table 5.2.2.3  
Weight of Spiral Reinforcement**

O.D. SPIRAL (in)	WEIGHTS IN POUNDS PER FOOT OF HEIGHT			
	<sup>3</sup> / <sub>8</sub> " DIA. ROD		<sup>1</sup> / <sub>2</sub> " DIA. ROD	
	6" PITCH (lb/ft)	F (lb)	3" PITCH (lb/ft)	F (lb)
24	4.72	7.1	16.79	12.60
26	5.12	7.7	18.19	13.65
28	5.51	8.3	19.59	14.70
30	5.91	8.9	20.99	15.75
32	6.30	9.5	22.38	16.80
34	6.69	10.1	23.78	17.85
36	7.09	10.7	25.18	18.90
38	7.48	11.2	26.58	20.00
40	7.87	11.8	27.98	21.00
42	8.27	12.4	29.38	22.00
44	8.66	13.0	30.78	23.10
46	9.06	13.6	32.18	24.10
48	9.45	14.2	33.58	25.20
50	9.84	14.8	34.98	26.20
52	10.24	15.4	36.38	27.30
54	10.63	15.9	37.77	28.30
56	11.02	16.5	39.17	29.40
58	11.42	17.1	40.57	30.40
60	11.81	17.7	41.97	31.50
62	12.21	18.3	43.37	32.50
64	12.60	18.9	44.77	33.60
66	12.99	19.5	46.17	34.60
68	13.39	20.1	47.57	35.70

For more complete coverage, see *CRSI Design Handbook*.

Total weight = (wt. per ft x height) + F

F = weight to add for finishing

(this includes 1<sup>1</sup>/<sub>2</sub> turns at the top and 1<sup>1</sup>/<sub>2</sub> turns at the bottom of spiral)

For additional information see MnDOT 2472 and AASHTO LRFD 5.10.6.2

TENSION LAP SPLICES FOR EPOXY COATED BARS WITH >12" CONCRETE CAST BELOW

$f_y=60$  ksi  $f_c=4$  ksi

Conc. Cover	Bar Size	Reinforcement Bar Spacing																
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"		
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	
2"	3	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	
	4	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	
	5	2'-7"	3'-4"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	
	6	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	
	7	3'-11"	5'-1"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	
	8	5'-2"	6'-8"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	
	9	6'-6"	8'-6"	5'-3"	6'-9"	5'-1"	6'-7"	5'-1"	6'-7"	5'-1"	6'-7"	5'-1"	6'-7"	5'-1"	6'-7"	5'-1"	6'-7"	
	10	8'-3"	10'-9"	6'-7"	8'-7"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"	6'-3"	8'-2"	
	11	10'-2"	13'-3"	8'-2"	10'-7"	7'-6"	9'-9"	7'-6"	9'-9"	7'-6"	9'-9"	7'-6"	9'-9"	7'-6"	9'-9"	7'-6"	9'-9"	
	14	N/A	N/A	11'-9"	15'-3"	10'-8"	13'-10"	10'-4"	13'-5"	10'-4"	13'-5"	10'-4"	13'-5"	10'-4"	13'-5"	10'-4"	13'-5"	
	2 3/8"	3	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"
		4	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
		5	2'-7"	3'-4"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"
		6	3'-1"	4'-0"	3'-1"	4'-0"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"
7		3'-11"	5'-1"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	
8		5'-2"	6'-8"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	
9		6'-6"	8'-6"	5'-3"	6'-9"	4'-9"	6'-2"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	
10		8'-3"	10'-9"	6'-7"	8'-7"	6'-0"	7'-10"	5'-6"	7'-2"	5'-6"	7'-2"	5'-6"	7'-2"	5'-6"	7'-2"	5'-6"	7'-2"	
11		10'-2"	13'-3"	8'-2"	10'-7"	7'-5"	9'-8"	6'-10"	8'-10"	6'-8"	8'-7"	6'-8"	8'-7"	6'-8"	8'-7"	6'-8"	8'-7"	
14		N/A	N/A	11'-9"	15'-3"	10'-8"	13'-10"	9'-9"	12'-9"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	
≥ 3"		3	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"
		4	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
		5	2'-7"	3'-4"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"
		6	3'-1"	4'-0"	3'-1"	4'-0"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"	2'-10"	3'-8"
	7	3'-11"	5'-1"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-4"	4'-4"	3'-4"	4'-4"	3'-4"	4'-4"	3'-4"	4'-4"	
	8	5'-2"	6'-8"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	
	9	6'-6"	8'-6"	5'-3"	6'-9"	4'-9"	6'-2"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	
	10	8'-3"	10'-9"	6'-7"	8'-7"	6'-0"	7'-10"	5'-6"	7'-2"	5'-3"	6'-9"	5'-3"	6'-9"	5'-3"	6'-9"	5'-3"	6'-9"	
	11	10'-2"	13'-3"	8'-2"	10'-7"	7'-5"	9'-8"	6'-10"	8'-10"	6'-3"	8'-2"	5'-10"	7'-7"	5'-10"	7'-6"	5'-10"	7'-6"	
	14	N/A	N/A	11'-9"	15'-3"	10'-8"	13'-10"	9'-9"	12'-9"	9'-0"	11'-9"	8'-5"	10'-11"	7'-10"	10'-2"	7'-8"	9'-11"	

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete, and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of $A_s$ spliced within required lap length	
$A_{s, provided}/A_{s, required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where:  $A_{s, provided}$  = Area of reinforcement provided and  $A_{s, required}$  = Area of reinforcement required by analysis

Figure 5.2.2.1  
Reinforcement Data

TENSION LAP SPLICES FOR EPOXY COATED BARS WITH ≤ 12" CONCRETE CAST BELOW

$f_y=60$  ksi  $f_c=4$  ksi

Conc. Cover	Bar Size	Reinforcement Bar Spacing																
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"		
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	
1"	3	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	1'-5"	1'-10"	
	4	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	
	5	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	2'-9"	3'-6"	
	6	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	
	7	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	
	8	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	6'-0"	7'-10"	
	9	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	7'-4"	9'-7"	
	10	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	8'-11"	11'-7"	
	11	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	10'-6"	13'-8"	
	14	N/A	N/A	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	14'-0"	18'-2"	
	1 1/2"	3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
		4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
		5	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
		6	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"
7		3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	3'-7"	4'-8"	
8		4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	4'-6"	5'-11"	
9		5'-9"	7'-6"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	5'-7"	7'-3"	
10		7'-4"	9'-6"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	6'-10"	8'-11"	
11		9'-0"	11'-8"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	8'-2"	10'-7"	
14		N/A	N/A	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	11'-0"	14'-4"	
2"		3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
		4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
		5	2'-3"	3'-0"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
		6	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"
	7	3'-6"	4'-6"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	
	8	4'-6"	5'-11"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	
	9	5'-9"	7'-6"	4'-7"	6'-0"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"	4'-6"	5'-10"	
	10	7'-4"	9'-6"	5'-10"	7'-7"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	
	11	9'-0"	11'-8"	7'-2"	9'-4"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"	6'-8"	8'-8"	
	14	N/A	N/A	10'-4"	13'-5"	9'-5"	12'-3"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	9'-1"	11'-10"	
	2 3/8"	3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
		4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
		5	2'-3"	3'-0"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
		6	2'-9"	3'-7"	2'-9"	3'-7"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"
7		3'-6"	4'-6"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	
8		4'-6"	5'-11"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	
9		5'-9"	7'-6"	4'-7"	6'-0"	4'-2"	5'-5"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	
10		7'-4"	9'-6"	5'-10"	7'-7"	5'-4"	6'-11"	4'-11"	6'-4"	4'-10"	6'-4"	4'-10"	6'-4"	4'-10"	6'-4"	4'-10"	6'-4"	
11		9'-0"	11'-8"	7'-2"	9'-4"	6'-7"	8'-6"	6'-0"	7'-10"	5'-7"	7'-2"	5'-2"	6'-8"	5'-1"	6'-8"	5'-1"	6'-8"	
14		N/A	N/A	10'-4"	13'-5"	9'-5"	12'-3"	8'-8"	11'-3"	8'-1"	10'-5"	8'-1"	10'-5"	8'-1"	10'-5"	8'-1"	10'-5"	
≥ 3"		3	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"	1'-1"	1'-5"
		4	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
		5	2'-3"	3'-0"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
		6	2'-9"	3'-7"	2'-9"	3'-7"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"	2'-2"	2'-10"
	7	3'-6"	4'-6"	3'-2"	4'-2"	3'-2"	4'-2"	3'-2"	4'-2"	2'-7"	3'-4"	2'-7"	3'-4"	2'-7"	3'-4"	2'-7"	3'-4"	
	8	4'-6"	5'-11"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	3'-8"	4'-9"	2'-11"	3'-9"	2'-11"	3'-9"	2'-11"	3'-9"	
	9	5'-9"	7'-6"	4'-7"	6'-0"	4'-2"	5'-5"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	4'-1"	5'-4"	
	10	7'-4"	9'-6"	5'-10"	7'-7"	5'-4"	6'-11"	4'-11"	6'-4"	4'-7"	6'-0"	4'-7"	6'-0"	4'-7"	6'-0"	4'-7"	6'-0"	
	11	9'-0"	11'-8"	7'-2"	9'-4"	6'-7"	8'-6"	6'-0"	7'-10"	5'-7"	7'-2"	5'-2"	6'-8"	5'-1"	6'-8"	5'-1"	6'-8"	
	14	N/A	N/A	10'-4"	13'-5"	9'-5"	12'-3"	8'-8"	11'-3"	8'-0"	10'-4"	7'-5"	9'-7"	6'-11"	9'-0"	6'-9"	8'-9"	

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete, and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of $A_s$ spliced within required lap length	
$A_{s, provided}/A_{s, required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where:  $A_{s, provided}$  = Area of reinforcement provided and  $A_{s, required}$  = Area of reinforcement required by analysis

Figure 5.2.2.2  
Reinforcement Data

TENSION LAP SPLICES FOR PLAIN UNCOATED BARS WITH >12" CONCRETE CAST BELOW

$f_y=60$  ksi  $f_c=4$  ksi

Conc. Cover	Bar Size	Reinforcement Spacing																
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"		
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	
2"	3	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	
	4	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	
	5	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	
	6	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	
	7	3'-0"	3'-11"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	
	8	3'-11"	5'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	
	9	5'-0"	6'-6"	4'-0"	5'-2"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	
	10	6'-4"	8'-3"	5'-1"	6'-7"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	4'-10"	6'-3"	
	11	7'-10"	10'-1"	6'-3"	8'-1"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"	5'-9"	7'-6"	
	14	N/A	N/A	9'-0"	11'-8"	8'-2"	10'-7"	7'-11"	10'-3"	7'-11"	10'-3"	7'-11"	10'-3"	7'-11"	10'-3"	7'-11"	10'-3"	
	≥ 3"	3	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"
		4	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"	1'-7"	2'-1"
		5	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
		6	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"	2'-5"	3'-1"
7		3'-0"	3'-11"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	
8		3'-11"	5'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	3'-2"	4'-1"	
9		5'-0"	6'-6"	4'-0"	5'-2"	3'-8"	4'-9"	3'-7"	4'-7"	3'-7"	4'-7"	3'-7"	4'-7"	3'-7"	4'-7"	3'-7"	4'-7"	
10		6'-4"	8'-3"	5'-1"	6'-7"	4'-7"	6'-0"	4'-3"	5'-6"	4'-0"	5'-2"	4'-0"	5'-2"	4'-0"	5'-2"	4'-0"	5'-2"	
11		7'-10"	10'-1"	6'-3"	8'-1"	5'-8"	7'-4"	5'-3"	6'-9"	4'-10"	6'-3"	4'-6"	5'-10"	4'-5"	5'-9"	4'-5"	5'-9"	
14		N/A	N/A	9'-0"	11'-8"	8'-2"	10'-7"	7'-6"	9'-9"	6'-11"	9'-0"	6'-5"	8'-4"	6'-0"	7'-10"	5'-10"	7'-7"	

TENSION LAP SPLICES FOR PLAIN UNCOATED BARS WITH ≤ 12" CONCRETE CAST BELOW

$f_y=60$  ksi  $f_c=4$  ksi

Conc. Cover	Bar Size	Reinforcement Spacing																
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"		
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	
2"	3	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	
	4	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	
	5	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	
	6	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	
	7	2'-4"	3'-0"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	
	8	3'-0"	3'-11"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	
	9	3'-10"	5'-0"	3'-1"	4'-0"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	
	10	4'-11"	6'-4"	3'-11"	5'-1"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	3'-9"	4'-10"	
	11	6'-0"	7'-10"	4'-10"	6'-3"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	
	14	N/A	N/A	6'-11"	9'-0"	6'-4"	8'-2"	6'-1"	7'-11"	6'-1"	7'-11"	6'-1"	7'-11"	6'-1"	7'-11"	6'-1"	7'-11"	
	≥ 3"	3	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"	11"	1'-3"
		4	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"	1'-3"	1'-7"
		5	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
		6	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"	1'-10"	2'-5"
7		2'-4"	3'-0"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	2'-2"	2'-9"	
8		3'-0"	3'-11"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	2'-5"	3'-2"	
9		3'-10"	5'-0"	3'-1"	4'-0"	2'-10"	3'-8"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	2'-9"	3'-7"	
10		4'-11"	6'-4"	3'-11"	5'-1"	3'-7"	4'-7"	3'-3"	4'-3"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	
11		6'-0"	7'-10"	4'-10"	6'-3"	4'-5"	5'-8"	4'-0"	5'-3"	3'-9"	4'-10"	3'-5"	4'-6"	3'-5"	4'-5"	3'-5"	4'-5"	
14		N/A	N/A	6'-11"	9'-0"	6'-4"	8'-2"	5'-9"	7'-6"	5'-4"	6'-11"	4'-11"	6'-5"	4'-8"	6'-0"	4'-6"	5'-10"	

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of $A_s$ spliced within required lap length	
$A_{s, provided}/A_{s, required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where:  $A_{s, provided}$  = Area of reinforcement provided and  $A_{s, required}$  = Area of reinforcement required by analysis

Figure 5.2.2.3  
Reinforcement Data

TENSION LAP SPLICES FOR STAINLESS STEEL BARS WITH >12" CONCRETE CAST BELOW

$f_y=75$  ksi  $f_c'=4$  ksi

Conc. Cover	Bar Size	Reinforcement Bar Spacing																
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"		
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	
2"	3	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	
	4	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	
	5	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	
	6	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	
	7	3'-9"	4'-11"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	
	8	4'-11"	6'-5"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"
	9	6'-3"	8'-1"	5'-0"	6'-6"	4'-7"	5'-11"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	
	10	7'-11"	10'-3"	6'-4"	8'-3"	5'-9"	7'-6"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	
	11	9'-9"	12'-8"	7'-10"	10'-1"	7'-1"	9'-2"	6'-6"	8'-5"	6'-0"	7'-10"	5'-7"	7'-3"	5'-6"	7'-2"	5'-6"	7'-2"	
	14	N/A	N/A	11'-3"	14'-7"	10'-2"	13'-3"	9'-4"	12'-2"	8'-8"	11'-3"	8'-0"	10'-5"	7'-6"	9'-9"	7'-4"	9'-6"	
	2 3/8"	3	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
		4	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
		5	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"
		6	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"
7		3'-9"	4'-11"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	
8		4'-11"	6'-5"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"
9		6'-3"	8'-1"	5'-0"	6'-6"	4'-7"	5'-11"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	
10		7'-11"	10'-3"	6'-4"	8'-3"	5'-9"	7'-6"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	5'-3"	6'-10"	
11		9'-9"	12'-8"	7'-10"	10'-1"	7'-1"	9'-2"	6'-6"	8'-5"	6'-0"	7'-10"	5'-7"	7'-3"	5'-6"	7'-2"	5'-6"	7'-2"	
14		N/A	N/A	11'-3"	14'-7"	10'-2"	13'-3"	9'-4"	12'-2"	8'-9"	11'-4"	8'-9"	11'-4"	8'-9"	11'-4"	8'-9"	11'-4"	
≥ 3"		3	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"	1'-6"	1'-11"
		4	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"	2'-0"	2'-7"
		5	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"	2'-6"	3'-3"
		6	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"	3'-0"	3'-10"
	7	3'-9"	4'-11"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	3'-5"	4'-6"	
	8	4'-11"	6'-5"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"	5'-1"	3'-11"
	9	6'-3"	8'-1"	5'-0"	6'-6"	4'-7"	5'-11"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	4'-5"	5'-9"	
	10	7'-11"	10'-3"	6'-4"	8'-3"	5'-9"	7'-6"	5'-3"	6'-10"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	
	11	9'-9"	12'-8"	7'-10"	10'-1"	7'-1"	9'-2"	6'-6"	8'-5"	6'-0"	7'-10"	5'-7"	7'-3"	5'-6"	7'-2"	5'-6"	7'-2"	
	14	N/A	N/A	11'-3"	14'-7"	10'-2"	13'-3"	9'-4"	12'-2"	8'-8"	11'-3"	8'-0"	10'-5"	7'-6"	9'-9"	7'-4"	9'-6"	

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of $A_s$ spliced within required lap length	
$A_{s, provided}/A_{s, required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where:  $A_{s, provided}$  = Area of reinforcement provided and  $A_{s, required}$  = Area of reinforcement required by analysis

Figure 5.2.2.4  
Reinforcement Data

TENSION LAP SPLICES FOR STAINLESS STEEL BARS WITH ≤ 12" CONCRETE CAST BELOW

$f_y = 75 \text{ ksi}$        $f_c' = 4 \text{ ksi}$

Conc. Cover	Bar Size	Reinforcement Bar Spacing															
		4"		5"		5 1/2"		6"		6 1/2"		7"		7 1/2"		≥ 8"	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
1"	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"	2'-3"	2'-11"
	6	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"	3'-1"	4'-0"
	7	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"	4'-0"	5'-3"
	8	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"	5'-0"	6'-6"
	9	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"	6'-2"	8'-0"
10	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	7'-5"	9'-8"	
11	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	8'-9"	11'-5"	
14	N/A	N/A	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	11'-8"	15'-2"	
1 1/2"	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	8	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"
	9	4'-10"	6'-3"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"	4'-8"	6'-1"
10	6'-1"	7'-11"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	5'-8"	7'-5"	
11	7'-6"	9'-9"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	6'-10"	8'-10"	
14	N/A	N/A	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	9'-2"	11'-11"	
2"	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	2'-11"	3'-9"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"
	8	3'-9"	4'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	9	4'-10"	6'-3"	3'-10"	5'-0"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"	3'-9"	4'-11"
10	6'-1"	7'-11"	4'-11"	6'-4"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	4'-8"	6'-0"	
11	7'-6"	9'-9"	6'-0"	7'-10"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	5'-7"	7'-2"	
14	N/A	N/A	8'-8"	11'-3"	7'-10"	10'-2"	7'-7"	9'-10"	7'-7"	9'-10"	7'-7"	9'-10"	7'-7"	9'-10"	7'-7"	9'-10"	
2 3/8"	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	2'-11"	3'-9"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"
	8	3'-9"	4'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	9	4'-10"	6'-3"	3'-10"	5'-0"	3'-6"	4'-7"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"
10	6'-1"	7'-11"	4'-11"	6'-4"	4'-5"	5'-9"	4'-1"	5'-3"	4'-1"	5'-3"	4'-1"	5'-3"	4'-1"	5'-3"	4'-1"	5'-3"	
11	7'-6"	9'-9"	6'-0"	7'-10"	5'-6"	7'-1"	5'-0"	6'-6"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"	4'-11"	6'-4"	
14	N/A	N/A	8'-8"	11'-3"	7'-10"	10'-2"	7'-2"	9'-4"	6'-9"	8'-9"	6'-9"	8'-9"	6'-9"	8'-9"	6'-9"	8'-9"	
≥ 3"	3	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"	1'-2"	1'-6"
	4	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"	1'-6"	2'-0"
	5	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"	1'-11"	2'-6"
	6	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"	2'-3"	3'-0"
	7	2'-11"	3'-9"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"	2'-8"	3'-5"
	8	3'-9"	4'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"	3'-0"	3'-11"
	9	4'-10"	6'-3"	3'-10"	5'-0"	3'-6"	4'-7"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"	3'-5"	4'-5"
10	6'-1"	7'-11"	4'-11"	6'-4"	4'-5"	5'-9"	4'-1"	5'-3"	3'-10"	5'-0"	3'-10"	5'-0"	3'-10"	5'-0"	3'-10"	5'-0"	
11	7'-6"	9'-9"	6'-0"	7'-10"	5'-6"	7'-1"	5'-0"	6'-6"	4'-8"	6'-0"	4'-4"	5'-7"	4'-3"	5'-6"	4'-3"	5'-6"	
14	N/A	N/A	8'-8"	11'-3"	7'-10"	10'-2"	7'-2"	9'-4"	6'-8"	8'-8"	6'-2"	8'-0"	5'-9"	7'-6"	5'-8"	7'-4"	

Table includes modification factors for reinforcement location, epoxy coating, normal weight concrete and reinforcement confinement as specified in AASHTO Articles 5.11.2.1.2 and 5.11.2.1.3. Reinforcement confinement is conservatively calculated by taking transverse reinforcement index as 0. Excess reinforcement factor is taken conservatively as 1.0. Tension lap splice lengths are based on AASHTO Article 5.11.5.3.1. Concrete cover is defined as the cover to the bar being considered. For concrete cover or bar spacing that falls between table values, conservatively use lap splice shown in the table for smaller concrete cover or bar spacing.

TENSION LAP SPLICES	Percent of $A_s$ spliced within required lap length	
$A_{s, provided} / A_{s, required}$	≤ 50	> 50
≥ 2	Class A	Class B
< 2	Class B	Class B

Where:  $A_{s, provided}$  = Area of reinforcement provided and  $A_{s, required}$  = Area of reinforcement required by analysis

Figure 5.2.2.5  
Reinforcement Data

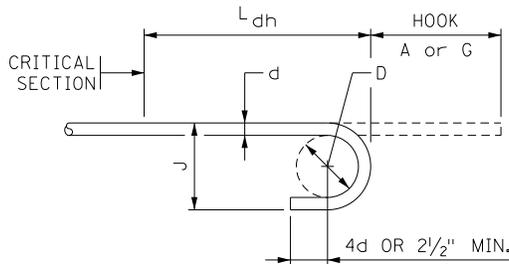
**DEVELOPMENT LENGTH FOR STANDARD HOOKS IN TENSION**

For plain and epoxy bars,  $f_y = 60$  ksi  $f'_c = 4$  ksi  
 For stainless steel bars,  $f_y = 75$  ksi

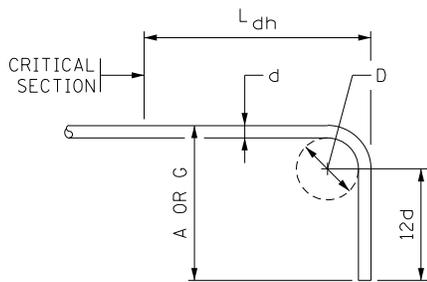
<b>Hooked Bar Development Length, <math>L_{dh}</math></b> When: Side Cover $\geq 2.5"$ AND For 90° Hooks, Concrete Cover $\geq 2"$ in the direction of bar extension			
Bar Size	Plain Uncoated Bars	Epoxy Coated Bars	Stainless Steel Bars
3	6"	7"	8"
4	8"	10"	10"
5	10"	1'-0"	1'-0"
6	1'-0"	1'-2"	1'-3"
7	1'-2"	1'-4"	1'-5"
8	1'-4"	1'-7"	1'-7"
9	1'-6"	1'-9"	1'-10"
10	1'-8"	2'-0"	2'-1"
11	1'-10"	2'-2"	2'-3"
14	2'-9"	3'-3"	3'-5"

<b>Hooked Bar Development Length, <math>L_{dh}</math></b> When: Side Cover $< 2.5"$ OR For 90° Hooks, Concrete Cover $< 2"$ in the direction of bar extension			
Bar Size	Plain Uncoated Bars	Epoxy Coated Bars	Stainless Steel Bars
3	8"	9"	9"
4	10"	1'-0"	1'-0"
5	1'-0"	1'-3"	1'-3"
6	1'-3"	1'-6"	1'-6"
7	1'-5"	1'-8"	1'-9"
8	1'-7"	1'-11"	2'-0"
9	1'-10"	2'-2"	2'-3"
10	2'-1"	2'-5"	2'-7"
11	2'-3"	2'-9"	2'-10"
14	2'-9"	3'-3"	3'-5"

Tables include modification factors per LRFD Art. 5.11.2.4.2 for normal weight concrete, bar coating, and reinforcement confinement. The reinforcement confinement factor is not applicable to bars larger than No. 11 bars. Note that MnDOT allows use of No. 14 bar standard hooks for concrete strengths up to 10 ksi.



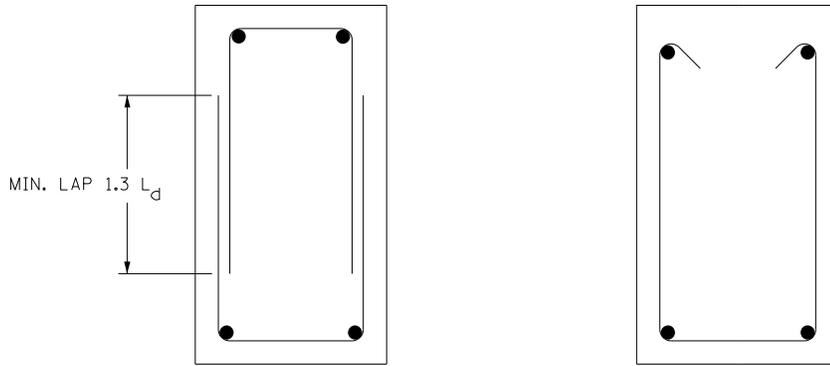
180° BEND



90° BEND

STANDARD HOOK DIMENSIONS				
BAR SIZE	D	180° HOOKS		90° HOOKS
		A OR G	J	A OR G
3	2 1/4"	5"	3"	6"
4	3"	6"	4"	8"
5	3 3/4"	7"	5"	10"
6	4 1/2"	8"	6"	1'-0"
7	5 1/4"	10"	7"	1'-2"
8	6"	11"	8"	1'-4"
9	9 1/2"	1'-3"	11 3/4"	1'-7"
10	10 3/4"	1'-5"	1'-1 1/4"	1'-10"
11	12"	1'-7"	1'-2 3/4"	2'-0"
14	18 1/4"	2'-3"	1'-9 3/4"	2'-7"

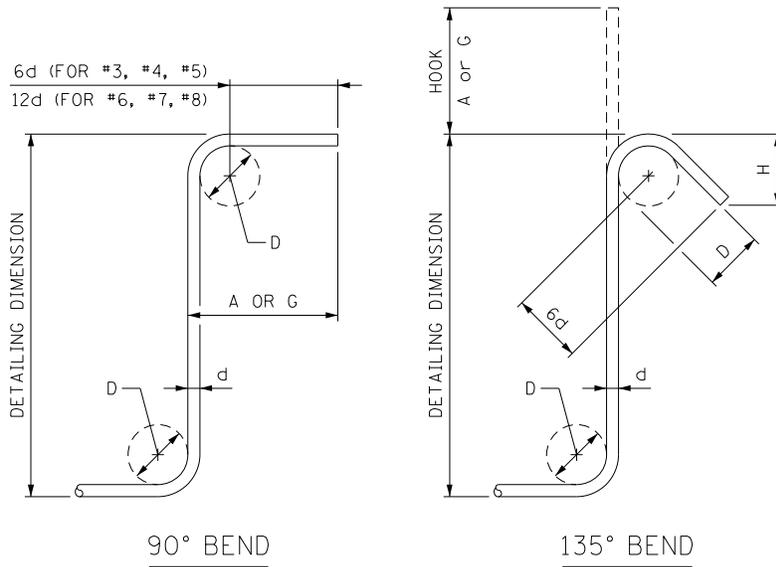
**Figure 5.2.2.6**  
**Reinforcement Data**



$L_d$  = DEVELOPMENT LENGTH

METHODS FOR ANCHORAGE OF SHEAR REINFORCEMENT

SEE AASHTO LRFD 5.11.2.6.2 AND 5.11.2.6.4



90° BEND

135° BEND

STIRRUP AND TIE HOOK DIMENSIONS				
BAR SIZE	D	90° HOOKS		135° HOOKS
		A OR G	A OR G	H *
3	1 1/2"	4"	4"	2 1/2"
4	2"	4 1/2"	4 1/2"	3"
5	2 1/2"	6"	5 1/2"	3 3/4"
6	4 1/2"	1'-0"	8"	4 1/2"
7	5 1/4"	1'-2"	9"	5 1/4"
8	6"	1'-4"	10 1/2"	6"

MINIMUM HORIZONTAL BAR SPACING (℄ TO ℄)		
BAR SIZE	CONCRETE MIX: 1G52 AND 3B52	CONCRETE MIX: 3JM, 3YHPC-M, 3YHPC-S, 3YLCHPC-M, 3YLCHPC-S, 3Y42, 3S52, 3Y82 AND 3W82
	3	2 5/8"
4	2 3/4"	2"
5	2 7/8"	2 1/8"
6	3"	2 1/4"
7	3 1/8"	2 3/8"
8	3 1/4"	2 1/2"
9	3 1/2"	2 7/8"
10	3 5/8"	3 1/4"
11	3 3/4"	3 5/8"
14	4 1/4"	4 1/4"

SEE AASHTO LRFD 5.10.3.1

NOTE: MINIMUM HORIZONTAL BAR SPACING SHALL ALSO APPLY TO THE DISTANCE FROM A CONTACT LAP SPLICE TO ADJACENT SPLICES OR BARS.

Figure 5.2.2.7  
Reinforcement Data

### **5.2.3 General Reinforcement Practices**

Reinforcement practices follow those shown by the Concrete Reinforcing Steel Institute (C.R.S.I.) in the *Manual of Standard Practice*. These practices include:

- 1) For bent bars, omit the last length dimension on reinforcement bar details.
- 2) Use standard length bars for all but the last bar in long bar runs.
- 3) Use a maximum length of 60 feet for #4 deck or slab bars and 40 feet for other applications.
- 4) Use a maximum length of 60 feet for bars #5 and larger.
- 5) Recognize that bar cutting and bending tolerances are  $\pm 1$  inch for bars and that this tolerance is important for long straight bars that do not have lap splices to provide dimensional flexibility. Refer to MnDOT document *Suggested Reinforcement Detailing Practices*, which is located at <http://www.dot.state.mn.us/bridge/standards.html>, for more guidance on rebar detailing to account for tolerances.
- 6) Reinforcement bars longer than 60 feet or larger than #11 are available only on special order, and should be avoided. Designers should check with the State Bridge Design Engineer before using special order sizes or lengths.

### **5.2.4 Reinforcement Bar Couplers**

Reinforcement bar couplers are expensive compared to conventional lap splices. Where lap splices cannot be readily used (bridge widening projects, staged construction, large river pier longitudinal bars—anywhere that the available space for a rebar projection is limited), couplers should be considered. Where possible, stagger reinforcement bar couplers in order to distribute the stiffness of the couplers. There are numerous coupler types and sizes. For members that require couplers, consider the coupler outside diameter and length when detailing reinforcement, in order to avoid congestion and clear cover issues.

### **5.2.5 Adhesive Anchors**

Similar to bar couplers, adhesive anchors are expensive. Adhesive anchors are typically used to attach secondary structural members to new concrete or primary structural members to existing (old) concrete. A typical use is to attach a metal rail to a concrete base.

See Article 13.3.2 of this manual for an adhesive anchor design example.

Adhesive anchors shall not be used for constant tension applications.

**5.2.6 Shrinkage and Temperature Reinforcement [5.10.8]**

Follow the requirements for shrinkage and temperature reinforcement given in LRFD 5.10.8. An exception to this is that shrinkage and temperature reinforcement is not required in buried footings of typical bridges.

**5.3 Concrete Slabs**

In many bridge engineering documents the terms “concrete slab” and “concrete deck” are used interchangeably. Within this manual, “concrete slab” will refer to a superstructure type without supporting beam elements. In most cases, the primary reinforcement for slabs is parallel to the centerline of roadway. Likewise, within this manual “concrete decks” will refer to the superstructure element placed on top of beams or girders. In most cases, the primary reinforcement for a concrete deck is transverse to the centerline of roadway. Practices for concrete decks are described in Section 9 of this manual.

**5.3.1 Geometry**

The maximum span lengths for concrete slabs are as follows:

Number of Spans	Without Haunches	With Haunches
1	30 ft	40 ft
2	40 ft	50 ft
3 or 4	50 ft	60 ft

End spans should be approximately 80% of the center span length to balance moments and prevent uplift.

LRFD Table 2.5.2.6.3-1 provides guidance for recommended minimum structure depth as a function of span length for slab superstructures without haunches.

When haunches are required, use linear haunches in accordance with the following:

$$\text{Minimum slab depth at pier} = 1.33 \cdot \left[ \frac{S+10}{30} \right]$$

(includes wear course if present)

$$\text{Minimum slab depth in non-haunched area} = 0.8 \cdot \left[ \frac{S+10}{30} \right]$$

(includes wear course if present)

$$\text{Minimum haunch length } L = 0.15 \cdot S$$

(where S is the length of longest span)

**5.3.2****Design/Analysis**

Skew can be ignored for slab bridges with skew angles of 20° or less.

- Place transverse reinforcement parallel to substructures.

For slab bridges with skew angles between 20° and 45°

- Perform a two-dimensional plate analysis.
- Place transverse reinforcement normal to the bridge centerline.

Slab type bridges are not allowed for bridges with skew angles greater than 45°.

Slab bridges curved in plan may be designed as if straight. Designers should consider and investigate the need for providing additional reinforcement in the portion of the slab outside of chord lines connecting substructure units.

Do not include the concrete wearing course in section properties when performing strength and serviceability checks. This will ensure that the slab has adequate capacity if traffic is carried on the bridge during operations associated with milling off the old wearing course and the placement of a new wearing course. An exception to this is when checking the top reinforcement in the negative moment region for flexural crack control.

**[5.7.3.4]**

When checking crack control for slabs and decks, use the Class 2 exposure condition ( $\gamma_e = 0.75$ ). Although the actual clear cover may exceed 2 inches for the slab/deck top bars, calculate  $d_c$  using a maximum clear concrete cover equal to 2 inches.

Determine reinforcement bar cutoff points based on strength, serviceability, and minimum reinforcement requirements.

**[5.14.4.1]**

Although not required by AASHTO, MnDOT requires a check of one-way shear in slab bridges. For determination of the live load distribution factor for shear, assume that the live load is distributed over the entire width of the superstructure. Load all lanes and use the appropriate multiple presence factor. For determination of factored shear resistance, use  $\beta = 2.0$ . If shear reinforcement is needed, try thickening the slab to eliminate it. If shear reinforcement must be used, calculate the appropriate  $\beta$  and  $\sigma$  values using LRFD Article 5.8.3.4.2.

**5.3.3 Exterior Strip  
[4.6.2.1.4b]**

Outside edges of slab bridges contain the exterior strip or edge beam. At a minimum, the exterior strip reinforcement must match that of the interior portions of the bridge.

Special consideration for the design of edge beams is required for bridges with sidewalks. Separately poured sidewalks may be considered to act compositely with the slab when adequate means of shear transfer at the interface is provided.

**5.3.4  
Reinforcement  
Layout**

Use the following guidelines for layout of reinforcement in a simple span slab bridge (see example in Figure 5.3.4.1):

Interior strip reinforcement

- Top longitudinal – 1 spacing, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Exterior strip reinforcement

- Top longitudinal – 1 spacing, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Transverse reinforcement – 1 spacing, 1 bar size

Use the following guidelines for layout of reinforcement in a continuous slab bridge:

Option 1 (see example in Figure 5.3.4.2):

Interior strip reinforcement

- Top longitudinal – 2 spacings, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Exterior strip reinforcement

- Top longitudinal – 2 spacings, 1 bar size
- Bottom longitudinal – 2 spacings, 1 bar size

Transverse reinforcement – 1 spacing, 1 bar size

Option 2 (see example in Figure 5.3.4.3):

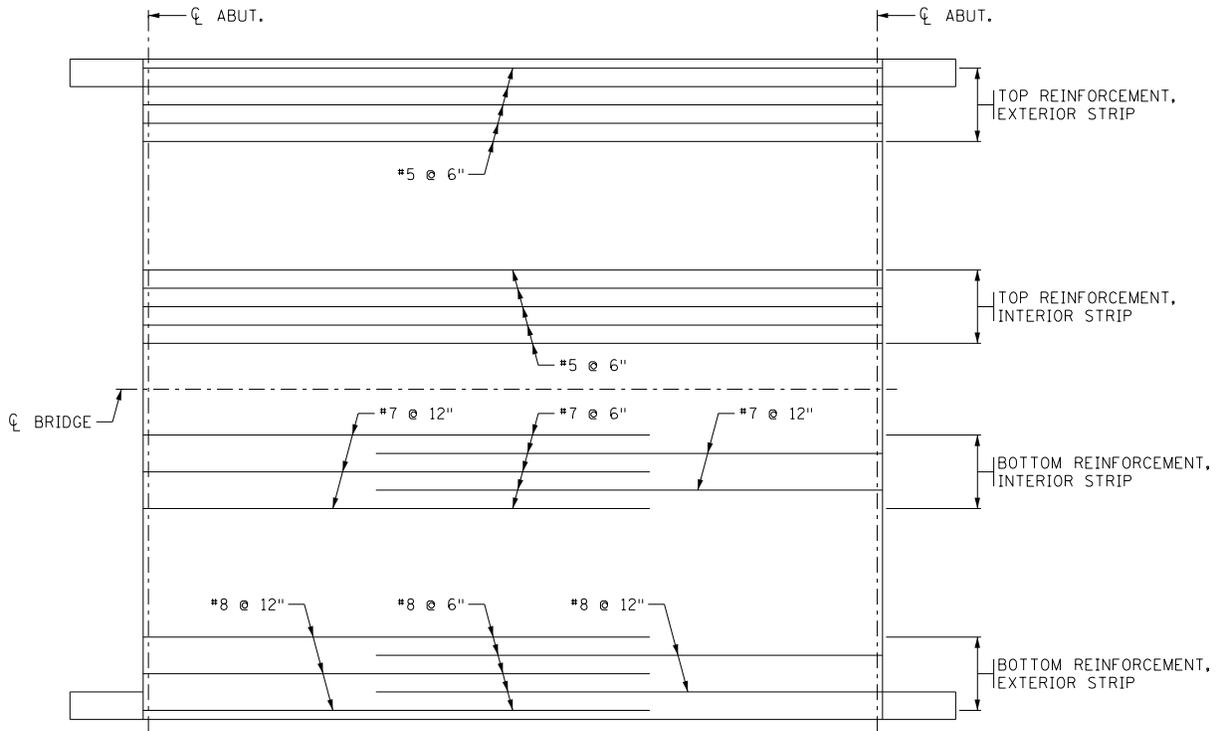
Interior strip reinforcement

- Top longitudinal – 2 spacings, 2 bar sizes
- Bottom longitudinal – 2 spacings, 2 bar sizes

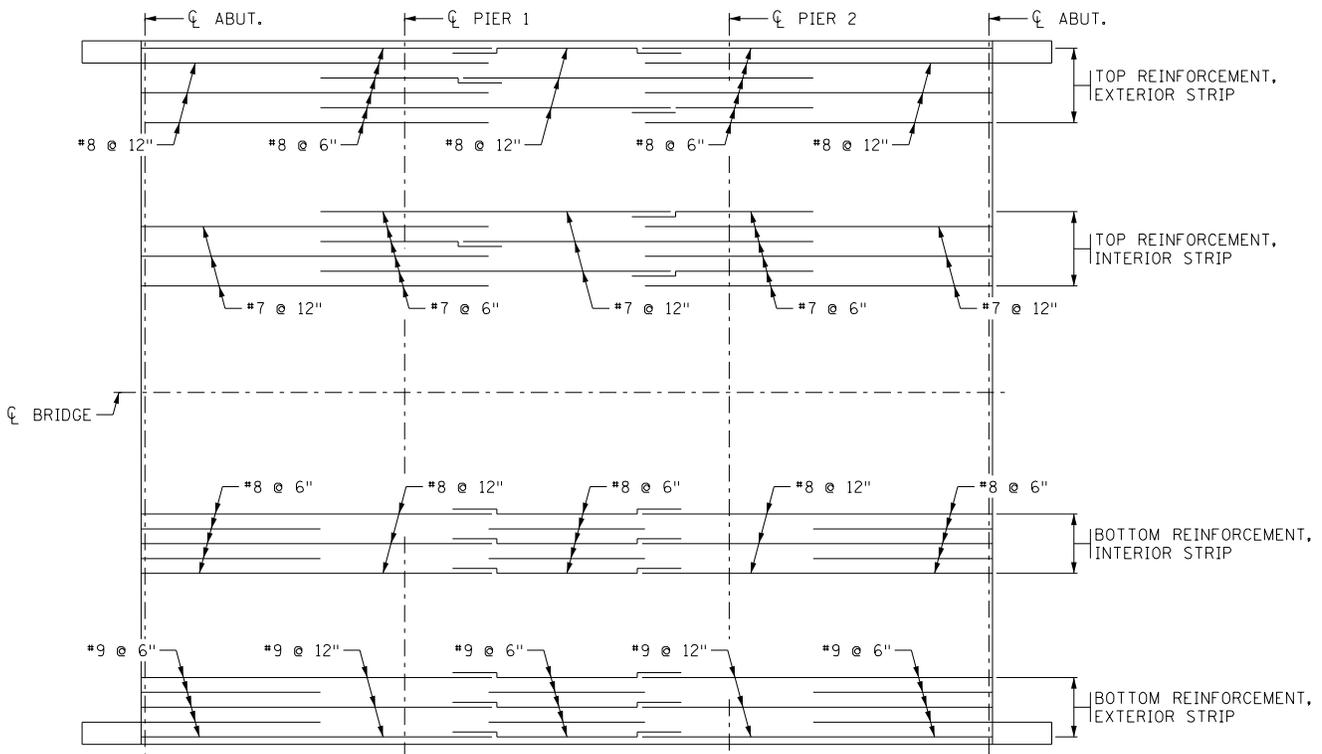
Exterior strip reinforcement

- Top longitudinal – 2 spacings, 2 bar sizes
- Bottom longitudinal – 2 spacings, 2 bar sizes

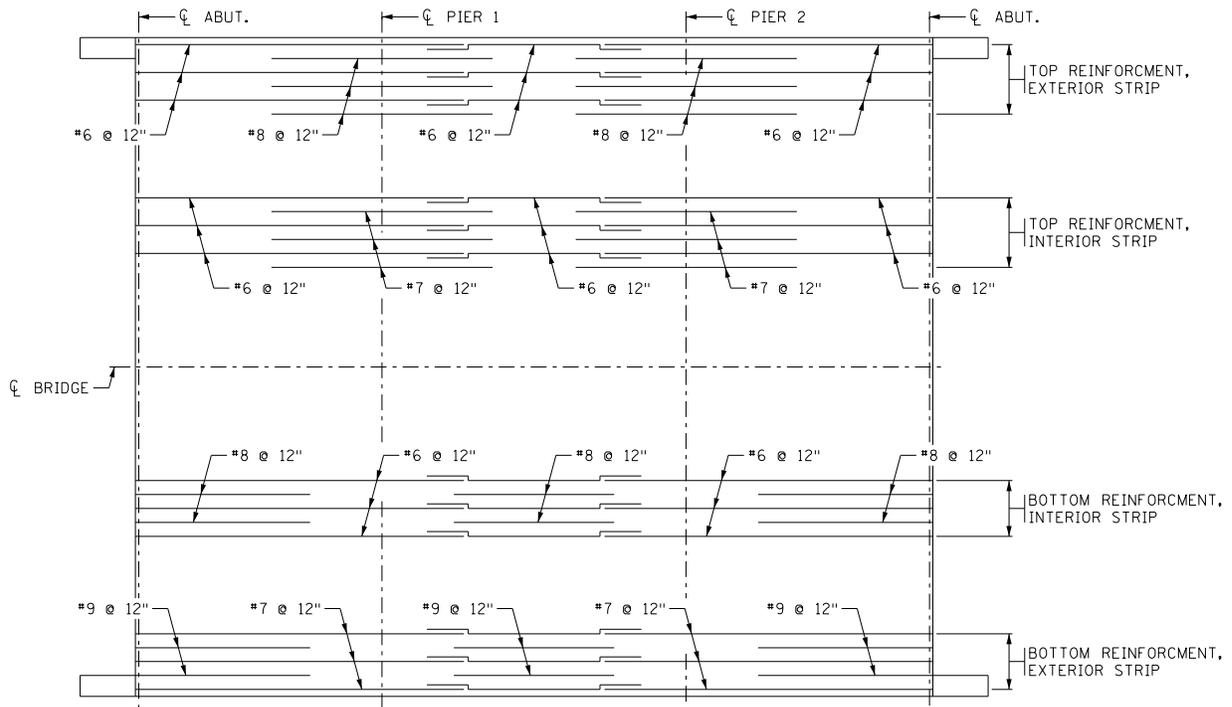
Transverse reinforcement - 1 spacing, 1 bar size



**Figure 5.3.4.1**



**Figure 5.3.4.2**



**Figure 5.3.4.3**

To simplify placement, detail reinforcement such that top bars are positioned over bottom bars where possible. For example, if the design requires bottom longitudinal bars spaced on 10 inch centers, top longitudinal bars might be spaced on 10 inch centers in positive moment regions and 5 inch centers in negative moment regions.

Extend railing dowel bars to the bottom layer of slab reinforcement and provide a horizontal leg for ease of placement.

**5.3.5 Camber and Deflections**

**[5.7.3.6.2]**

In order to obtain the best rideability over the life of the structure, camber concrete slab bridges for the immediate dead load deflection plus one half of the long-term deflection. Use gross section properties for dead load deflection calculations and a long-term creep multiplier of 4.0.

Railings, sidewalks, medians, and wearing courses are not placed while the slab is supported on falsework. Assume that only the slab carries the dead load of these elements.

Check live load deflections using the effective moment of inertia. The effective moment of inertia may be approximated as one half of the gross moment of inertia. The maximum live load deflection is  $L/800$  for vehicular bridges that do not carry pedestrians and  $L/1000$  for vehicular bridges that carry pedestrians.

Consider the concrete wearing course to be functioning compositely with the slab for live load deflection. Assume the riding surface has lost  $1/2$  inch of thickness due to wear.

Use a live load distribution factor equal to the number of lanes times the multiple presence factor and divide by the width of the slab for the deflection check.

#### **5.4 Pretensioned Concrete**

The details of pretensioned concrete beams are presented on standard Bridge Details Part II sheets incorporated into a set of plans. Prepare a separate sheet for each type of beam in the project. Beams are identical if they have the same cross-section, strand layout, concrete strengths, and a similar length. To simplify fabrication and construction, try to minimize the number of beam types incorporated into a project. Design exterior beams with a strength equal to or greater than the interior beams.

##### **5.4.1 Geometry**

Provide a minimum stool along centerline of beam that is based on  $1\frac{1}{2}$  inches of minimum stool at edge of flange. For dead load computations assume an average stool height equal to the minimum stool height plus 1 inch. Deck cross slopes, horizontal curves, and vertical curves all impact the stool height.

There are several Bridge Office practices regarding the type and location of diaphragms or cross frames for prestressed beam bridges:

- 1) Design prestressed I-beam bridges without continuity over the piers, except in the following situations:
  - a) Bridge is over water with pile bent piers supported by unstable soils such as fat clay.
  - b) Bridge is over water with pile bent piers at risk for large ice or debris loading and pier does not have an encasement wall.
- 2) Intermediate diaphragms are not required for 14RB, 18RB, 22RB, and 27M beams. For all other beam sizes, the following applies. Intermediate diaphragms are not required for single spans of 45'-0" or less. Provide one diaphragm per every 45 feet of span length, spaced evenly along the span as stated in Table 5.4.1.1.

**Table 5.4.1.1**

<b>Span length (ft)</b>	<b>Base number of intermediate diaphragms</b>
Less than 45'-0"	0
45'-0" to 90'-0"	1 located at midspan
90'-0" to 135'-0"	2 located at the third points
135'-0" to 180'-0"	3 located at the quarter points
Greater than 180'-0"	4 plus an additional diaphragm for each additional 45 ft of span length greater than 180'-0"

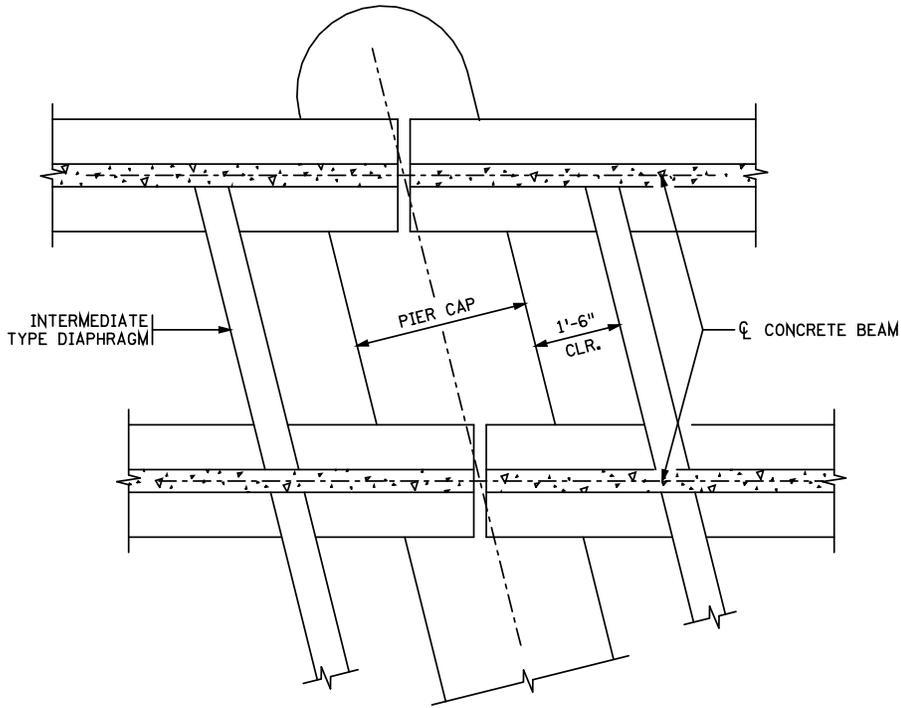
For spans over traffic, place additional diaphragms in the fascia bay approached by traffic to provide bracing against impact from over-height traffic loads. For two-lane roadways, place one diaphragm approximately over each shoulder. For additional lanes, space additional diaphragms at intervals of about 25'-0" over the roadway.

- 3) Figure 5.4.1.1 illustrates the typical layout of intermediate diaphragms at piers for bridges without continuity over the piers.

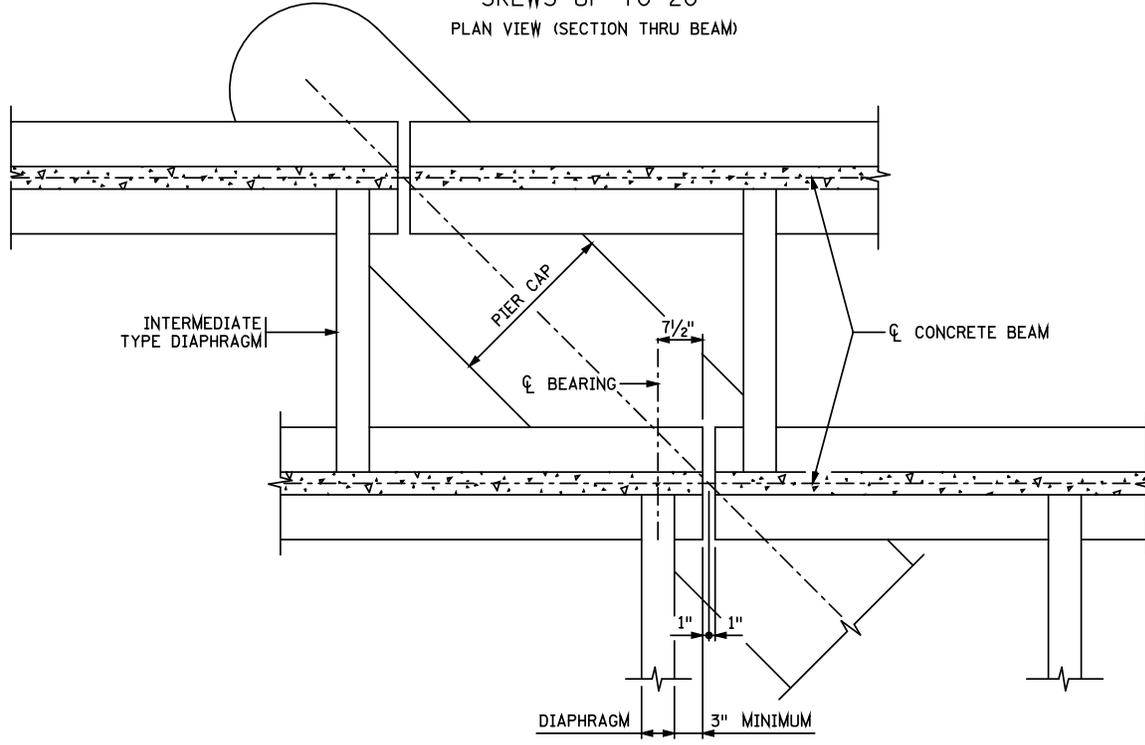
Locate the centerline of bearing  $7\frac{1}{2}$  inches from the end of the beam for RB, M, and MN shapes. Locate the centerline of bearing  $8\frac{1}{2}$  inches from the end of the beam for MW shapes. For MW shapes, this dimension can be adjusted if used with higher movement bearings, as opposed to the typical curved plate elastomeric bearings shown in Section 14 of this manual. However, if the  $8\frac{1}{2}$  inch dimension is exceeded, a special design for the bearing, sole plate, and beam end region must be completed.

At piers of two span bridges, provide 2 inches of clearance between the ends of RB, M, and MN beams. Provide 3 inches clearance for structures with three or more spans. Provide 4 inches of clearance between the ends of MW beams regardless of the number of spans. Note that the fabrication length tolerance for pretensioned I-beams is  $\pm\frac{1}{8}$ " per 10 feet of length. It may be necessary to cope beam flanges at piers for bridges with tight horizontal curves or at skewed abutments.

For bridges on significant grades ( $\geq 3\%$ ) the sloped length of the beam will be significantly longer than the horizontal length between substructure units. If the sloped length is  $\frac{1}{2}$  inch or more than the horizontal length, identify the sloped length dimension on the beam detail plan sheets.



SKEWS UP TO 20°  
PLAN VIEW (SECTION THRU BEAM)



SKEW OVER 20°  
PLAN VIEW (SECTION THRU BEAM)

**Figure 5.4.1.1**  
**Typical Diaphragm Layout at Piers for Prestressed Concrete Beam Bridge**  
**With Continuous Deck Over Piers**

**5.4.2 Stress Limits**  
**[5.9.3] [5.9.4]**

Similar to the Standard Specifications, the LRFD Specifications identify service load stress limits for different elements and locations.

For typical prestressed beams, check tension and compression service load stresses at two stages. The first stage is when the prestress force is transferred to the beams in the fabricator's yard. The second stage is after all losses have occurred and the beam is in the fully constructed bridge.

Design pretensioned beams with a maximum tension at transfer (after initial losses) of:

$$0.0948 \cdot \sqrt{f'_{ci}} \leq 0.2 \text{ ksi} \quad (\text{where } f'_{ci} \text{ is in ksi})$$

Design pretensioned beams with a maximum tension after all losses of:

$$0.19 \cdot \sqrt{f'_c} \quad (\text{where } f'_c \text{ is in ksi})$$

**5.4.3**  
**Design/Analysis**

Use the "approximate method" provided in LRFD Article 5.9.5.3 to compute prestress losses.

Design all pretensioned beams using uncoated low relaxation 0.6 inch ( $A_s = 0.217 \text{ in}^2$ ) diameter strands and epoxy coated mild reinforcement.

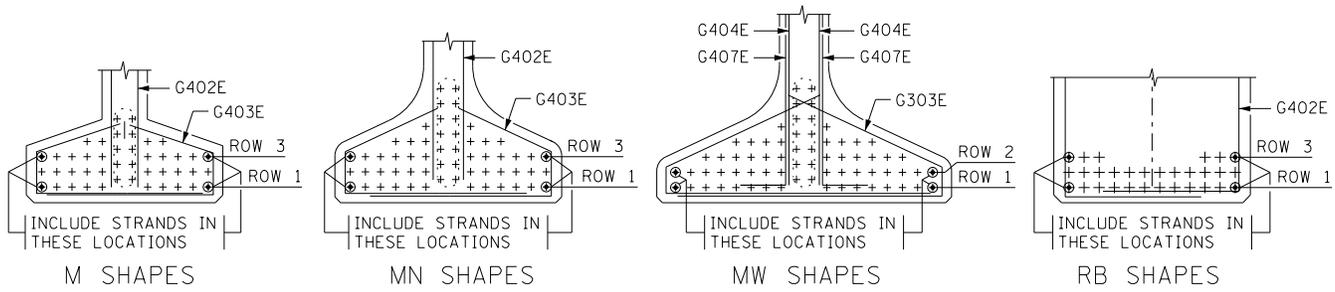
At the time of prestress transfer (initial), the minimum required concrete strength ( $f'_{ci}$ ) is 4.5 ksi and the maximum is limited to 7.5 ksi. At the termination of the curing period (final), the minimum concrete strength ( $f'_c$ ) is 5 ksi and the maximum strength is 9 ksi. Higher initial or final strengths may be used with approval from the Bridge Design Engineer. An initial concrete strength greater than 7 ksi may add cost to the beam. The fabricator cannot remove the beam from the bed until a cylinder break indicates the concrete has reached its specified initial strength. With strengths higher than 7 ksi, the fabricator may have to leave the beam in the bed longer than the normal 16-18 hours or add increased amounts of superplastizer and cement, thereby increasing the cost of the beam.

If possible, the initial concrete strength should be 0.5 to 1.0 ksi lower than the final concrete strength. Since concrete naturally gains strength with age, the final strength of the beam will be more efficiently utilized.

If the calculated initial or final strengths differ by more than 0.3 ksi from those used in the analysis, reanalyze the beam with the new values. Reanalysis is needed because changes to the concrete strengths  $f'_{ci}$  and  $f'_c$  affect the concrete modulus, which affects the prestress losses and the composite beam section modulus.

Straight strands must be arranged in a 2 inch grid pattern. See standard beam sheets for possible strand locations. Arrange draped strands in a 2 inch grid pattern independent of the straight strands. Use draped strands to reduce the initial required strength  $f'_{ci}$  at the end of the beam. Straight strands may be placed in the draped area at 2 inches from the bottom of the beam. Draped strands must start at 3 inches minimum from the bottom at the hold-downs and 3 inches minimum from the top at the end of the beam. Straight strands should be used in place of draped strands whenever possible.

In order to easily allow the fabricator to place and secure the stirrups in the bottom of the beam, always include strands in the outermost locations of rows 1 and 3 for RB, M, and MN shapes. Include strands in the outermost locations of rows 1 and 2 for MW shapes. Rows numbers are measured up from the bottom of the beam. See Figure 5.4.3.1.



**Figure 5.4.3.1**

Whenever possible, use a constant strand pattern for all girders on the same project. If the strand pattern varies between beams, the fabricator may be required to tension an entire bed length of strand in order to cast a single girder. This results in a large amount of wasted strand, and will increase the cost of the beam.

The maximum number of draped strands allowed at each hold-down point varies with the fabricator. Therefore, design and detail beams with one hold-down on each side of midspan, placed at 0.40L to 0.45L from the centerline of bearing. The fabricator will provide additional hold-downs as needed.

The following guidance is provided to designers to evaluate initial and final stresses to optimize their designs:

### **Final Stresses**

#### Midpoint Strength at Bottom of Beam

If tension stress is greater than  $0.19 \cdot \sqrt{f'_c}$  (0.570 ksi for 9 ksi concrete), lower the stress by:

- 1) Add 2 strands to the bottom row of straight and move the draped strands up 1 inch at midpoint (bottom row of draped at 4 inches).
- 2) Add 2 strands to the second row of straight and move the draped strands up 2 inches at midpoint (bottom row of draped at 6 inches) or add 2 draped strands (bottom row of draped at 4 inches).
- 3) Continue to add strands as stated above until 6 straight and 4 draped have been added. If the tension stress is still greater than 0.570 ksi, consider adding another line of beams to the bridge. If the tension stress is less than 0.570 ksi, two strands (either straight or draped) may be removed and the beam reanalyzed. If the stress becomes greater than 0.570 ksi, return to the original number of strands.

### **Initial Stresses**

#### Midpoint Strength at Bottom of Beam

If the required initial strength is greater than 7.0 ksi:

- 1) Move the center of gravity of the strands up at midpoint of the beam until either the final concrete strength becomes 9.0 ksi or the initial strength is 0.5 to 1.0 ksi lower than the final strength.
- 2) Remove 2 strands (preferably draped) from the beam and reanalyze. Keep in mind that changes will affect the required final strength. If the removing of strands increases the final concrete strength above 9.0 ksi, do not remove the strands but consider other changes in the strand pattern.

#### End Strength at Bottom of Beam

If the required initial strength is greater than 7.0 ksi and greater than that calculated at the midpoint:

- 1) Strands may be draped to decrease the required strength. Keep in mind that changes to strand locations at the end of the beam may affect the mid-beam stresses.
- 2) If the initial strength is lower than calculated at the midpoint, draped strands may be placed straight thereby decreasing the

hold-down force and the number of draped strands required. Keep in mind that changes may affect the mid-beam stresses.

#### End Strength at Top of Beam

If the required initial strength is greater than 7.0 ksi, raise the center of gravity of the strands at the end of the beam. This can be accomplished by draping strands that were previously straight or increasing the height of the draped strands.

#### Midpoint Strength at Top of Beam

If the required initial strength is greater than calculated at the bottom end or midpoint:

- 1) The center of gravity of the strands may be moved higher at the center.
- 2) The number of strands may be reduced to decrease the required strength.

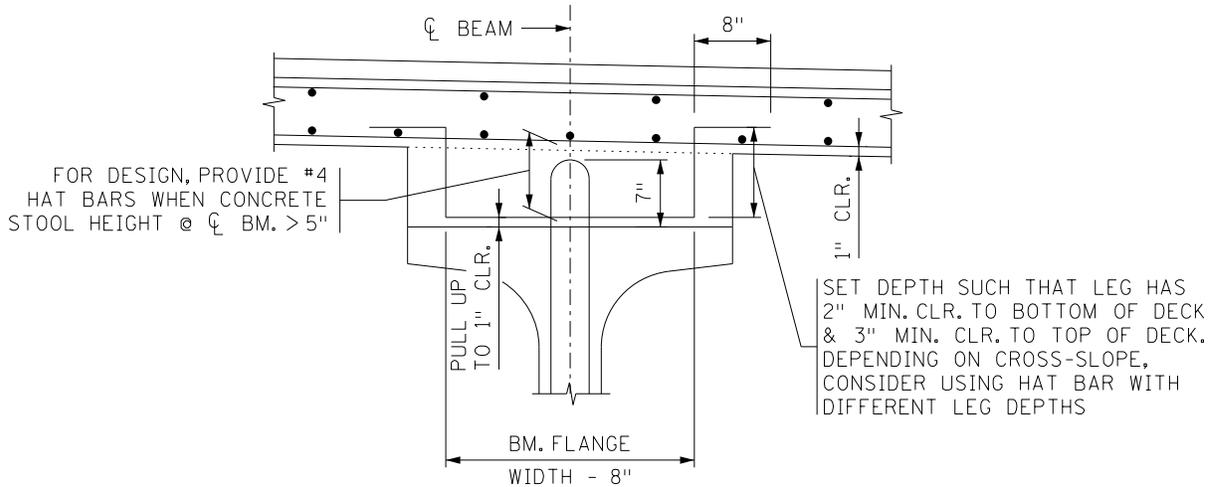
If the guidance above results in an initial concrete strength greater than 7.0 ksi, the initial strength may be increased up to a maximum value of 7.5 ksi. Note that this will likely increase the beam cost.

Ensure that adequate shear and splitting reinforcement is provided in the ends of beams. The maximum size for stirrup bars is #5. Based on the concrete mix used for prestressed beams, the minimum stirrup spacing is  $2\frac{1}{2}$  inches. If the required amount of reinforcement cannot be provided within  $\frac{h}{4}$  of the end of the beam, provide the remainder at a  $2\frac{1}{2}$  inch spacing.

Design shear reinforcement using the "General Procedure" provisions given in LRFD Article 5.8.3.4.2.

Horizontal shear reinforcement must be provided to ensure composite action between the beam and deck. MnDOT standard beam sheets accomplish this by extending the beam stirrups into the deck (G402E & G508E with 7" projection for RB, M, MN shapes and G404E & G508E with  $7\frac{1}{4}$ " projection for MW shapes). In order to ensure composite action, the shear reinforcement must extend into the deck far enough to engage the deck bottom mat of reinforcement. Check the stool heights over the length of the beams. For regions where stool heights are found greater than 5 inches at beam centerline, do not increase the stirrup length or pull up the stirrups, but rather provide #4 "hat" shaped bars as shown in Figure 5.4.3.2. Set the leg depth to provide 2 inches minimum clear to the bottom of deck and 3 inches clear to the top of deck for the upper hook, and 1 inch clear from the bottom of the bar to the beam flange. In

cases where field personnel report excessive stools not anticipated in the bridge plan, discuss with them whether one "hat" bar or two "Z" bars would be better for rebar placement.



HAT BAR FOR BEAMS WITH LARGE STOOL HEIGHTS

**Figure 5.4.3.2**

Due to the height of the "MW" series beams, investigate whether a deck pour sequence is needed to reduce the effects of beam end rotation on the end region of the deck.

**5.4.4 Detailing/ Reinforcement**

Identify the beam type on the beam sheet by depth in inches and length rounded to the next highest foot. In the superstructure quantities list, identify the beam type by depth. For example, an MN45 beam, 72'-4" long would be "MN45-73" on the beam sheet and "MN45" in the quantities list. Group beams of similar lengths with the same strand pattern into one type on a beam sheet. The pay item quantity will be the total length of beams (of each height) in feet.

**5.4.5 Camber and Deflection**

On the framing plan, show the beam and diaphragm spacing, staging, type of diaphragms, centerline of piers, centerline of abutment and pier bearings, working points, beam marks (B1, B2 etc.), the "X" end of beams, and the type and location of bearings. One end of each beam is labeled the "X" end after fabrication. This is used during erection to ensure that the beams are properly placed. Many times diaphragm inserts are not symmetric and beams can only be placed one way.

The standard beam sheets contain a camber diagram where designers are to provide camber information. Knowing the deflection values

associated with prestressing and different dead load components, camber values can be obtained.

MnDOT camber multipliers are used to approximately convert the prestress and selfweight deflections at the time of prestress transfer to the deflections at the time of erection. Use a camber multiplier of 1.40 for the prestress deflection component. Use a camber multiplier of 1.40 for the selfweight of the member. No multiplier is used for diaphragm dead loads, deck and stool dead loads or parapet and median dead loads. These camber multipliers differ from the PCI multipliers as they are based on research specific to MnDOT beams. They are based on a time lapse of 30 to 180 days between the time of prestress transfer and the time of beam erection for deck placement.

Use of the MnDOT camber multipliers does not apply to the "MW" series beams. Complete a refined camber analysis using an appropriate creep model for "MW" series camber determination. Then report the estimated camber values for various girder ages in the bridge plan.

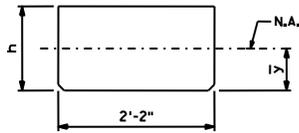
The "Erection Camber" is the camber of the beam at the time of erection after the diaphragms are in place. The "Est. Dead Load Deflection" is the sum of deflections associated with the placement of the deck, railings, sidewalks, and stool. Do not include the weight of the future wearing surface when computing the dead load deflection.

**5.4.6 Standard  
I-Beams**

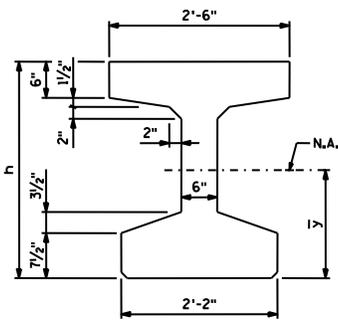
I-beam sections available for use in Minnesota include the "M" series, "MN" series, and "MW" series. The "M" series sections range in depth from 27 inches to 36 inches and have identical top and bottom flange dimensions along with a 6 inch thick web. The "MN" series sections range in depth from 45 inches to 63 inches. The "MN" series sections are more efficient than the "M" series due to wider top and bottom flanges and a  $6\frac{1}{2}$  inch thick web. Due to the development of the MN45, MN54, MN63, and 82MW shapes, most of the M shapes (45M, 54M, 63M, 72M, and 81M) have been archived. The 27M and 36M shapes continue to be available as there is no corresponding MN shape at those depths. The "MW" series sections allow for spans in the range of 150 to 200 feet. Figures 5.4.6.1 through 5.4.6.4 contain section properties and preliminary beam spacing vs. span length curves for all standard I-beam shapes.

**5.4.7 Rectangular  
Beams**

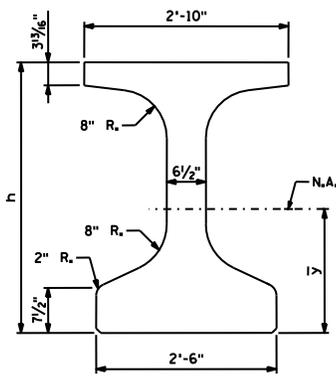
Solid rectangular prestressed beams may be used on short span bridges. These units are most appropriate for short span structures requiring a low profile or where construction of falsework for a slab structure would be difficult or unwanted. Figure 5.4.6.1 and 5.4.6.2 contain section properties and preliminary beam spacing vs. span length curves for the standard rectangular beams.



RECTANGULAR BEAM



"M" SERIES I-BEAM



"MN" SERIES I-BEAM

DESIGN ASSUMPTIONS FOR PRESTRESSED CONCRETE BEAM CHART:

2012 AASHTO LRFD Bridge Design Specifications, 6th Edition.

HL-93 Live Load

Beam Concrete:  $f'_c = 9.0$  ksi  $f'_{ci} = 7.5$  ksi  $w_{bm} = 0.155$  kips/ft<sup>3</sup>

$$E_c = 1265\sqrt{f'_c} + 1000 \text{ ksi}$$

Deck Concrete:  $f'_c = 4.0$  ksi  $E_c = 3644$  ksi

$$w_c = 0.145 \text{ kcf for } E_c \text{ computation}$$

$$w_c = 0.150 \text{ kcf for dead load computation}$$

0.6" diameter low relaxation strands,  $E_s = 28,500$  ksi

$$f_{pu} = 270 \text{ ksi with initial pull of } 0.75 f_{pu}$$

Simple supports with six beams and deck without wearing course.  
Deck carries two F-Rails with no sidewalk or median, skew = 0 degrees.

Effective deck thickness is total deck thickness minus 1/2" of wear.

1 1/2" stool height used for composite beam section properties.  
2 1/2" average stool height used for dead load calculations.

Rail dead load applied equally to all beams.  
Dead load includes 0.020 ksf future wearing course.

Approximate long term losses are used per LRFD 5.9.5.3.

Service Concrete Tensile Stress Limits:

$$\text{After Initial Losses: } 0.094\sqrt{f'_{ci}} \leq 0.2 \text{ ksi}$$

$$\text{After All Losses: } 0.19\sqrt{f'_c}$$

Beam Properties

BEAM	h (in)	SHAPE	AREA (in <sup>2</sup> )	W ① (lb/ft)	$\bar{y}$ (in)	I (in <sup>4</sup> )	S <sub>B</sub> (in <sup>3</sup> )	A <sub>c</sub> ② (in <sup>2</sup> )
14RB	14	Rect.	364	392	7.00	5,945	849	312
18RB	18	Rect.	468	504	9.00	12,640	1,404	364
22RB	22	Rect.	572	616	11.00	23,070	2,097	416
27M	27	I-Beam	516	555	13.59	43,080	3,170	296
36M	36	I-Beam	570	614	17.96	93,530	5,208	323
MN45	45	I-Beam	690	743	20.58	178,780	8,687	427
MN54	54	I-Beam	749	806	24.63	285,230	11,580	457
MN63	63	I-Beam	807	869	28.74	421,750	14,670	486

① Based on 155 pounds per cubic foot.

② Based on a 9" slab with 1/2" of wear and 1 1/2" stool. See LRFD 5.8.3.4.2 for A<sub>c</sub> definition.

**Figure 5.4.6.1**  
**Precast Prestressed Concrete Beam Data (RB, M, MN)**

**PRESTRESSED CONCRETE BEAM CHART FOR RB, M, & MN SERIES**  
 (Chart is for preliminary use only. See Figure 5.4.6.1 for design assumptions.)

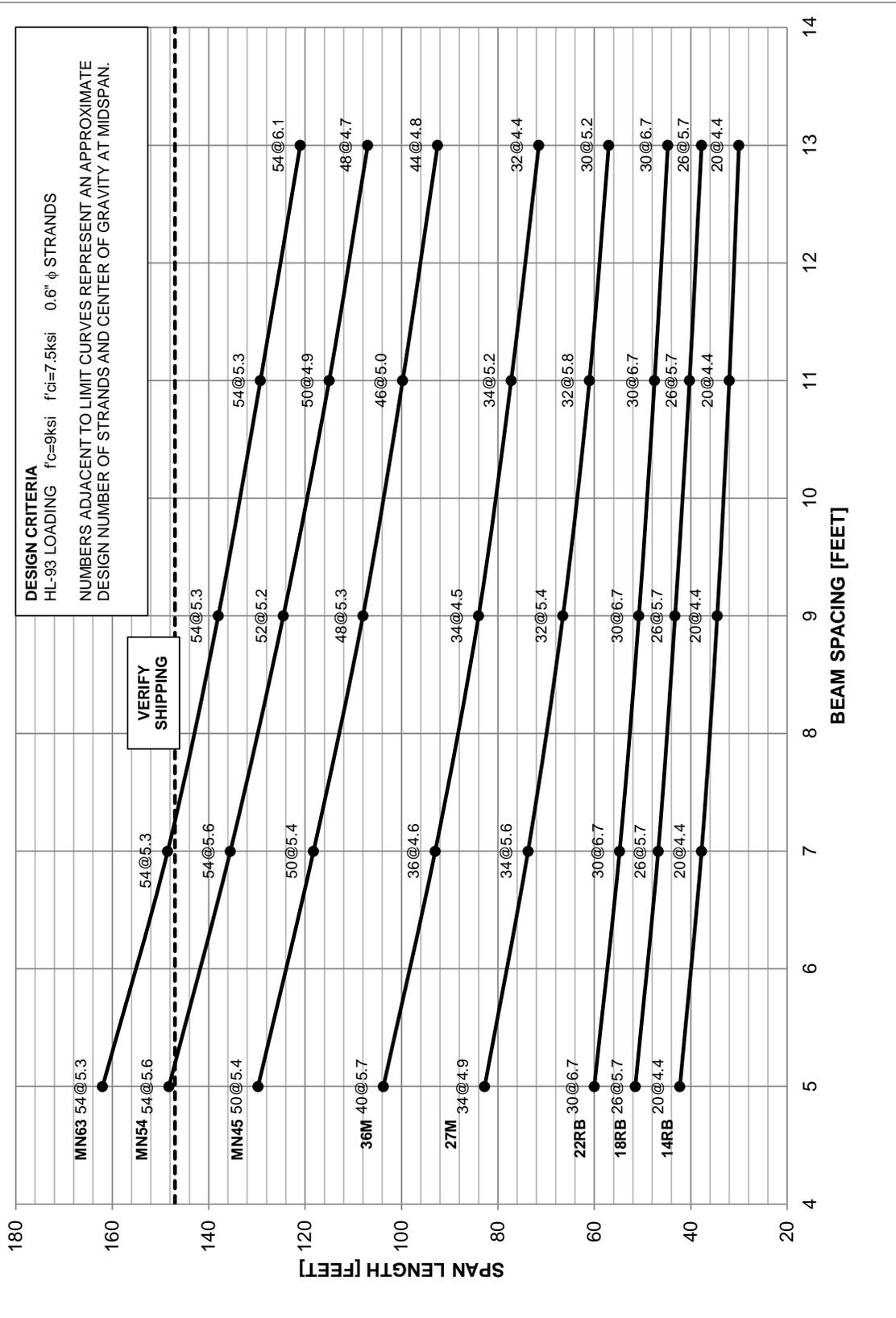
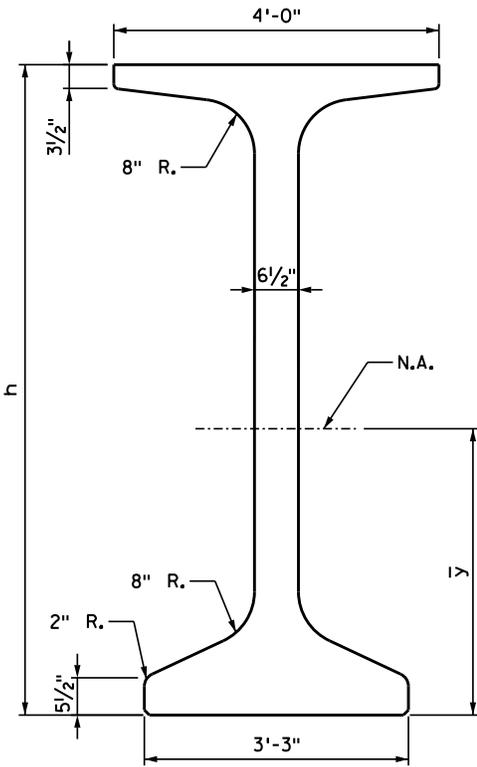


Figure 5.4.6.2



"MW" SERIES I-BEAM

DESIGN ASSUMPTIONS FOR PRESTRESSED CONCRETE BEAM CHART:

2012 AASHTO LRFD Bridge Design Specifications, 6<sup>th</sup> Edition.

HL-93 Live Load

Beam Concrete:  $f'_c = 9.0$  ksi  $f'_{ci} = 7.5$  ksi  $w_{bm} = 0.155$  kips/ft<sup>3</sup>

$$E_c = 1265\sqrt{f'_c} + 1000 \text{ ksi}$$

Deck Concrete:  $f'_c = 4.0$  ksi  $E_c = 3644$  ksi

$$w_c = 0.145 \text{ kcf for } E_c \text{ computation}$$

$$w_c = 0.150 \text{ kcf for dead load computation}$$

0.6" diameter low relaxation strands,  $E_s = 28,500$  ksi

$$f_{pu} = 270 \text{ ksi with initial pull of } 0.75 f_{pu}$$

Simple supports with six beams and deck without wearing course.

Deck carries two F-Rails with no sidewalk or median, skew = 0 degrees.

Effective deck thickness is total deck thickness minus 1/2" of wear.

1 1/2" stool height used for composite beam section properties.

2 1/2" average stool height used for dead load calculations.

Rail dead load applied equally to all beams.

Dead load includes 0.020 ksf future wearing course.

Approximate long term losses are used per LRFD 5.9.5.3.

Service Concrete Tensile Stress Limits:

$$\text{After Initial Losses: } 0.094\sqrt{f'_{ci}} \leq 0.2 \text{ ksi}$$

$$\text{After All Losses: } 0.19\sqrt{f'_c}$$

Beam Properties

BEAM	h (in)	SHAPE	AREA (in <sup>2</sup> )	W ① (lb/ft)	$\bar{y}$ (in)	I (in <sup>4</sup> )	S <sub>B</sub> (in <sup>3</sup> )	A <sub>c</sub> ② (in <sup>2</sup> )
82MW	82	I-Beam	1062	1143	38.37	1,010,870	26,345	609
96MW	96	I-Beam	1153	1241	45.02	1,486,510	33,019	655

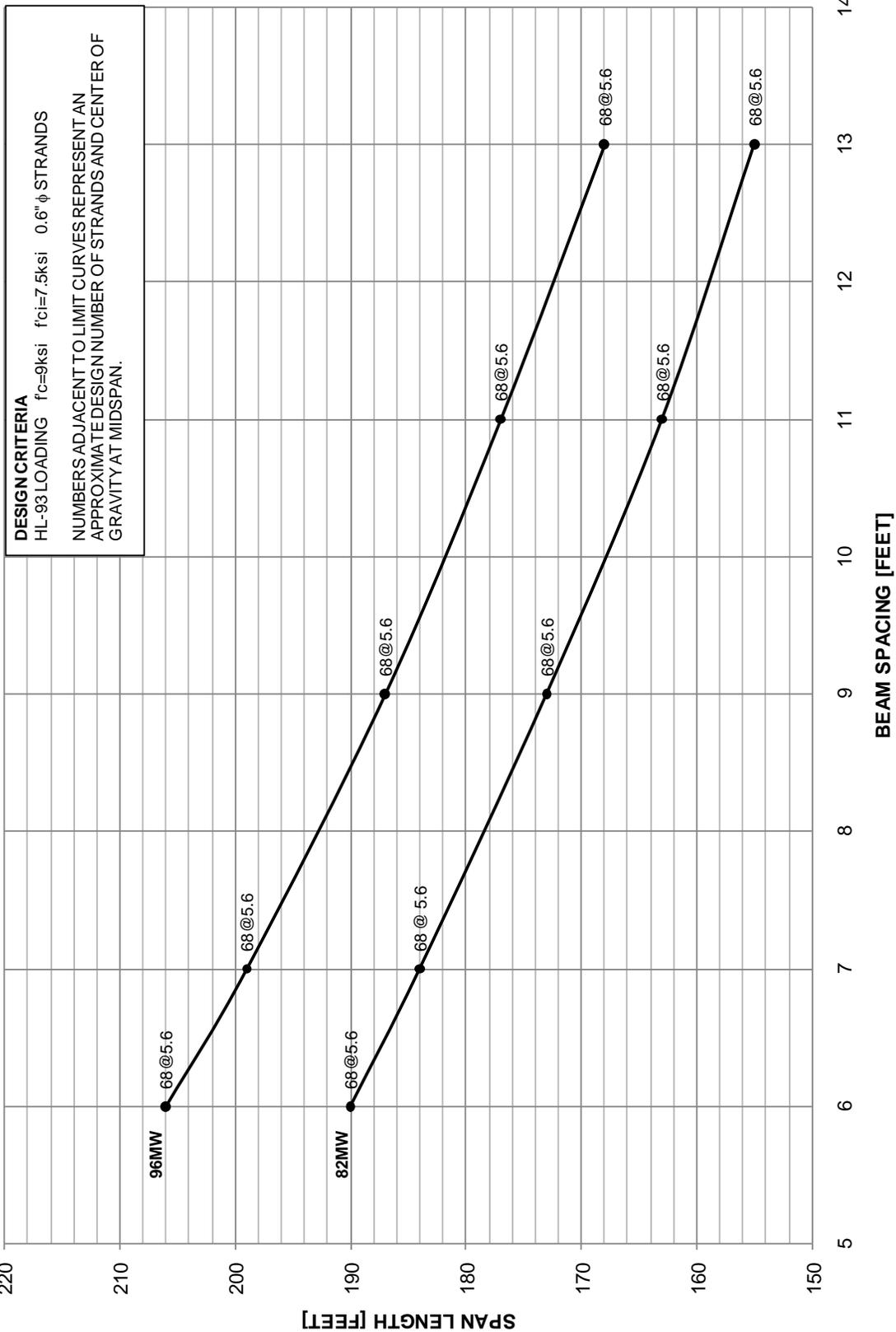
① Based on 155 pounds per cubic foot.

② Based on a 9" slab with 1/2" of wear and 1 1/2" stool. See LRFD 5.8.3.4.2 for A<sub>c</sub> definition.

Figure 5.4.6.3

Precast Prestressed Concrete Beam Data for MW Series

**PRESTRESSED CONCRETE BEAM CHART FOR MW SERIES**  
(Chart is for preliminary use only. See Figure 5.4.6.3 for design assumptions.)



**Figure 5.4.6.4**

**5.4.8  
Double-Tee Beams**

Pretensioned double-tees are not used anymore on Minnesota bridges. The standard Bridge Details Part II Figures 5-397.525 and 5-397.526 were archived in 2009.

**5.5  
Post-Tensioned  
Concrete**

Post-tensioned (PT) concrete structures have their prestressing steel stressed after the concrete has been placed and partially cured.

Post-tensioned concrete bridges are specialty structures. Poor detailing and poor construction practices can greatly reduce the service life of these structures. Designers should follow current practices recommended by the American Segmental Bridge Institute (ASBI) and the Post-Tensioning Institute (PTI).

Design segmental box girders and post-tensioned concrete slab bridges for zero tension under service loads.

**5.5.1 PT Slab  
Bridges**

Post-tensioned concrete slab bridges are used for projects requiring spans longer than those efficiently accommodated with conventionally reinforced concrete slabs. The drawback to post-tensioned slabs is that they are more complex to design and construct. Elastic shortening and secondary bending moments due to post-tensioning are important design parameters for post-tensioned slab bridges. During construction a number of additional components are involved. They include the installation of post-tensioning ducts and anchorages, the pushing or pulling of strands through the ducts, the jacking of tendons, and grouting operations.

**5.5.2  
PT I-Girders**

Post-tensioned spliced I-girder bridges are not commonly used in Minnesota, but the MW series beams were developed with consideration of future use for spliced girder bridges. MnDOT will develop appropriate details as potential projects are identified.

**5.5.3 PT Precast or  
Cast-In-Place Box  
Girders**

The depth of box girders should preferably be a minimum of  $1/18$  of the maximum span length.

Place vertical webs of box girders monolithic with the bottom slab.

**5.6 Concrete  
Finishes and  
Coatings**

The finish or coating to be used on concrete elements will usually be determined when the Preliminary Bridge Plan is assembled. In general, provide a finish or coating consistent with the guidance given in the *Aesthetic Guidelines for Bridge Design Manual*.

A wide variety of surface finishes for concrete are used on bridge projects. These range from plain concrete to rubbed concrete to painted surfaces to form liners and stains. Plain concrete and rubbed concrete finishes are described in the MnDOT Spec. 2401. Painted and architectural surfaces must be described in the special provisions.

Specify graffiti protection for concrete elements with a coating system that has more than one color.

**5.7 Design  
Examples**

Three design examples complete Section 5. The examples consist of a three-span reinforced concrete slab superstructure, a prestressed I-beam superstructure, and a three-span post-tensioned slab superstructure.

**5.7.1 Three-Span  
Haunched  
Reinforced  
Concrete Slab**

This example illustrates the design of a haunched reinforced concrete slab bridge. The three continuous spans are 44'-0", 55'-0", and 44'-0" in length. The roadway width is 44'-0" with MnDOT Type F barrier railings for a total out-to-out width of 47'-4". The bridge is skewed 10 degrees. A plan view and typical sections of the bridge are shown in Figures 5.7.1.1 and 5.7.1.2.

After determining live load distribution factors, dead and live loads are computed at span tenth points. Next the live load deflection and the shear capacity of the section is checked. Then using Strength I, Service I, and Fatigue design moments the flexural reinforcement is sized. This is accomplished by:

- Providing adequate steel for strength
- Verifying that crack control checks are satisfied
- Checking fatigue stresses in the reinforcement
- Verifying that minimum reinforcement checks are satisfied

Finally, distribution and shrinkage and temperature reinforcement is sized.

Material and design parameters used in this example are:

Concrete Strength at 28 Days,  $f'_c = 4.0$  ksi

Concrete Unit Weight,  $w_c = 0.150$  kcf (dead loads)

$w_c = 0.145$  kcf (modulus)

Reinforcing Bars:

Yield Strength,  $f_y = 60$  ksi

Modulus of Elasticity,  $E_s = 29,000$  ksi

Weight of Future Wearing Surface = 20 psf

Weight of concrete rail = 0.439 kip/ft

For simplicity, the wearing course is assumed to extend from out-to-out of deck.

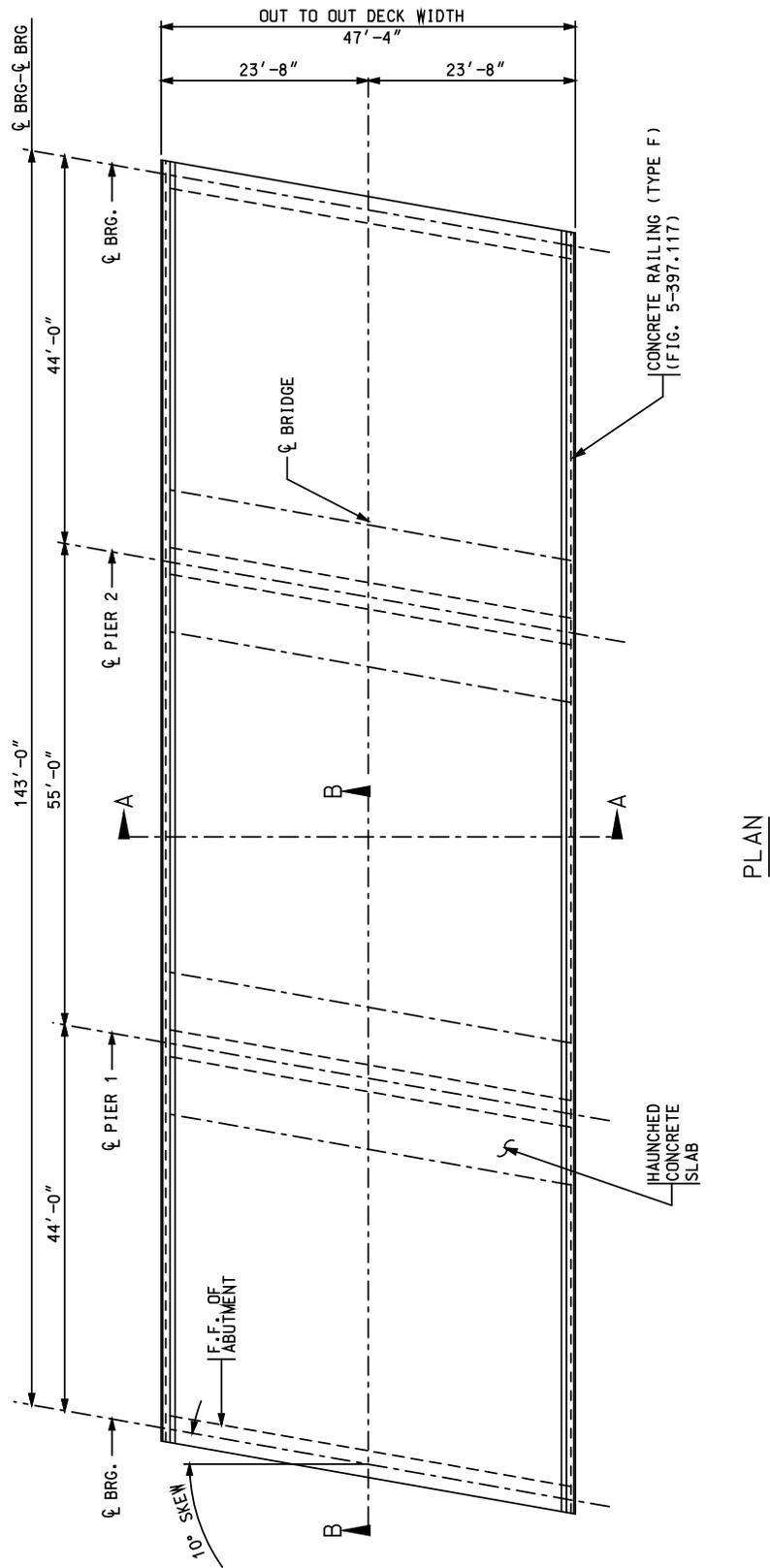
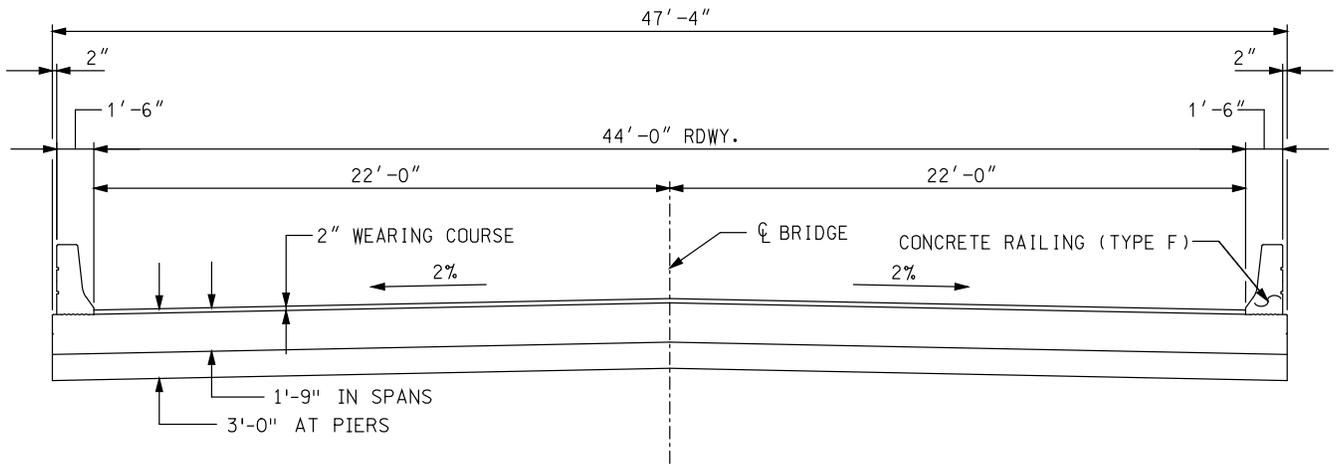
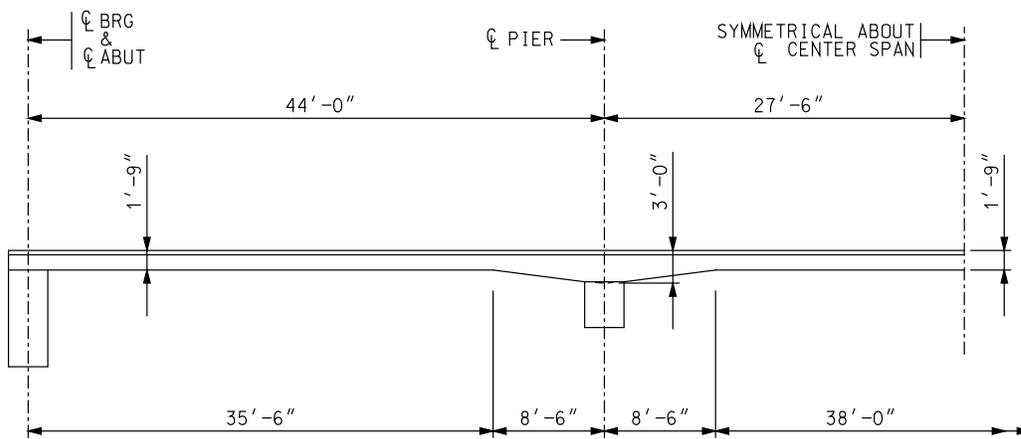


Figure 5.7.1.1



SECTION A-A



SECTION B-B

Figure 5.7.1.2

**A. Determine Slab Depths**

MnDOT's practice is to use linear haunches, with the haunch length equal to 15 % of the longest span.

$$\text{Haunch Length} = 0.15 \cdot 55 = 8.25 \text{ ft} \quad \text{Use } \underline{8.5 \text{ ft}}$$

The minimum slab depth at midspan ( $h_{\min}$ ) is also determined with the length of the longest span (S):

$$0.80 \cdot \frac{(S+10)}{30} = 0.80 \cdot \frac{(55+10)}{30} = 1.73 \text{ ft} \quad \text{Use } \underline{h_{\min} = 1.75 \text{ ft}}$$

The depth of the slab required at the pier ( $h_{\max}$ ) is determined with an equation based on the length of the longest span:

$$1.33 \cdot \frac{(S+10)}{30} = 1.33 \cdot \frac{(55+10)}{30} = 2.88 \text{ ft} \quad \text{Use } \underline{h_{\max} = 3.00 \text{ ft}}$$

The slab depth (h) includes the 2 inch wearing course.

**B. Determine Interior Live Load Strip Width**  
**[4.6.2.3]**  
**[3.6.1.1.1]**

The LRFD Specifications contain equations to determine the strip width that carries a lane of live load. Slab designs are performed on a strip one foot wide. The strip widths found with the LRFD equations are inverted to arrive at the live load distribution factor for a 1 foot wide strip (LLDF).

For interior strips multiple equations are evaluated to determine whether one or multiple live load lanes govern.

**Flexure – One Lane Loaded**

Multiple Presence Factors have been incorporated into the LRFD equations per LRFD C3.6.1.1.2.

**[Eqn. 4.6.2.3-1]**

$$\text{Equivalent strip width (in), } E = 10.0 + 5.0 \cdot \sqrt{L_1 \cdot W_1}$$

Where:

$L_1$  is the modified span length.

It is equal to the span length, but can be no greater than 60.

$W_1$  is the modified bridge width.

It is the minimum bridge width, but can be no greater than 30.

For the 44 ft side spans:

$$E = 10.0 + 5.0 \cdot \sqrt{44 \cdot 30} = 191.7 \text{ in/lane}$$

$$\text{Therefore } \text{LLDF}_{\text{SL}} = \frac{1}{\left(\frac{191.7}{12}\right)} = 0.063 \text{ lanes/ft} \quad \text{Governs}$$

For the 55 ft center span:

$$E = 10.0 + 5.0 \cdot \sqrt{55 \cdot 30} = 213.1 \text{ in/lane}$$

$$\text{Therefore } LLDF_{SL} = \frac{1}{\left(\frac{213.1}{12}\right)} = 0.056 \text{ lanes/ft}$$

### Flexure – Fatigue Loading

Divide the one lane LLDF by 1.2 to remove the Multiple Presence Factor

For the 44 ft side spans:

$$LLDF_{FAT} = \frac{0.063}{1.2} = 0.052 \text{ lanes/ft} \quad \underline{\text{Governs}}$$

For the 55 ft center span:

$$LLDF_{FAT} = \frac{0.056}{1.2} = 0.047 \text{ lanes/ft}$$

### Flexure – Multiple Lanes Loaded

**[Eqn. 4.6.2.3-2]**

$$\text{Equivalent strip width (in), } E = 84.0 + 1.44 \cdot \sqrt{L_1 \cdot W_1} \leq \frac{12 \cdot W}{N_L}$$

$$L_1 = 44 \text{ ft or } 55 \text{ ft}$$

$$W_1 = 47.33 \text{ ft}$$

W is the actual bridge width = 47.33 ft

$$N_L = \frac{44}{12} = 3.7 \quad \underline{\text{Use 3}}$$

The upper limit on the equivalent strip width is:

$$\frac{12.0 \cdot W}{N_L} = \frac{12.0 \cdot 47.33}{3} = 189.3 \text{ in/lane}$$

For the 44 ft side spans:

$$E = 84.0 + 1.44 \cdot \sqrt{44 \cdot 47.33} = 149.7 \leq 189.3 \text{ in/lane}$$

$$\text{Therefore the } LLDF_{ML} = \frac{1}{\left(\frac{149.7}{12}\right)} = 0.080 \text{ lanes/ft} \quad \underline{\text{Governs}}$$

For the 55 ft center span:

$$E = 84.0 + 1.44 \cdot \sqrt{55 \cdot 47.33} = 157.5 \leq 189.3 \text{ in/lane}$$

$$\text{Therefore the } LLDF_{ML} = \frac{1}{\left(\frac{157.5}{12}\right)} = 0.076 \text{ lanes/ft}$$

To simplify the process of arriving at design forces, the maximum distribution factor (0.080 lanes/ft) will be used for all locations.

### Shear and Deflection

#### [5.14.4.1]

Although not required by AASHTO, MnDOT requires that slab type bridges be checked for shear using the same live load distribution factor calculated for deflection.

#### [2.5.2.6.2]

All design lanes should be loaded and the entire slab assumed to resist the loads.

$$N_L = 3 \quad m = 0.85 \quad IM = 33\%$$

Dynamic load allowance (IM) is applied only to the truck portion of the live load. The distribution factor for the lane portion of the live load is:

$$LLDF_{s\Delta} = \frac{(\# \text{ of lanes}) \cdot (MPF)}{(\text{deck width})} = \frac{3 \cdot 0.85}{47.33} = 0.054 \text{ lanes/ft}$$

The distribution factor for the truck portion is:

$$LLDF_{s\Delta} \cdot (1 + IM) = 0.054 \cdot (1 + 0.33) = 0.072 \text{ lanes/ft}$$

### Reduction for Skew

#### [Eqn. 4.6.2.3-3]

$$r = 1.05 - 0.25 \cdot \tan\theta = 1.05 - 0.25 \cdot \tan(10^\circ) = 1.006$$

No Reduction

### C. Determine Exterior Live Load Strip Width

#### [4.6.2.1.4]

The exterior strip is assumed to carry one wheel line and a tributary portion of lane load.

Check if the equivalent strip is less than the maximum width of 72 inches.

$$E = (\text{Distance from edge to inside of barrier}) + 12 + \left(\frac{\text{smallest int. } E}{4}\right)$$

$$= 20 + 12 + \frac{149.7}{4} = 69.4 < 72.0 \text{ in} \quad \text{Use } \underline{69.4 \text{ in}}$$

Compute the distribution factor associated with one truck wheel line:

$$\begin{aligned} LLDF_{EXTT} &= \frac{1 \text{ wheelline} \cdot MPF}{(2 \text{ wheellines/lane}) \cdot (E/12)} = \frac{1 \cdot 1.2}{(2) \cdot (69.4/12)} \\ &= 0.104 \text{ lanes/ft} \end{aligned}$$

Compute the distribution factor associated with lane load on a 69.4 inch wide exterior strip:

$$\begin{aligned} LLDF_{EXTL} &= \frac{\left(\frac{\text{deckwidth loaded}}{10 \text{ ft loadwidth}}\right) \cdot MPF}{(\text{exteriorstrip width})} = \frac{\left(\frac{69.4/12 - 20/12}{10}\right) \cdot 1.2}{(69.4/12)} \\ &= 0.085 \text{ lanes/ft} \end{aligned}$$

For simplicity, the larger value (0.104 lanes/ft) is used for both load types when assembling design forces.

**D. Resistance Factors and Load Modifiers**  
**[5.5.4.2.1]**  
**[1.3.3-1.3.5]**

The following resistance factors will be used for this example:

- ϕ = 0.90 for flexure and tension (assumed, must be checked)
- ϕ = 0.90 for shear and torsion

The following load modifiers will be used for this example:

		Strength	Service	Fatigue
Ductility	η <sub>D</sub>	1.0	1.0	1.0
Redundancy	η <sub>R</sub>	1.0	1.0	1.0
Importance	η <sub>I</sub>	1.0	n/a	n/a
	η = η <sub>D</sub> · η <sub>R</sub> · η <sub>I</sub>	1.0	1.0	1.0

**E. Select Applicable Load Combinations and Load Factors**  
**[3.4.1]**

Three load combinations will be considered for the design example. STRENGTH I - Will be considered with a standard HL-93 loading.

$$U = 1.0 \cdot [1.25 \cdot (DC) + 1.75 \cdot (LL + IM)]$$

SERVICE I - Will be used primarily for crack control checks.

$$U = 1.0 \cdot (DC) + 1.0 \cdot (LL + IM)$$

FATIGUE - Will be used to evaluate the reinforcing steel.

$$U = 0.75 \cdot (LL + IM)$$

**F. Calculate Live Load Force Effects [3.6.1]**

The LRFD Specifications contain several live load components that are combined and scaled to generate design live loads. The components include: HL-93 design truck, lane loading, tandem axles, a truck train, and a fatigue truck with fixed axle spacings.

For this example the following combinations will be investigated:

Design Truck + Design Lane

Design Tandem + Design Lane

0.90 (Truck Train + Design Lane) (Neg. Moment Regions)

Fatigue Truck

The dynamic load allowance (IM) has the following values:

IM = 15% when evaluating fatigue and fracture.

IM = 33% when evaluating all other limit states.

It is not applied to the lane live load.

**G. Calculate Force Effects from Other Loads**

The dead load from the barriers is conservatively assumed to be fully carried by both interior and exterior strips. Since the slab thickness varies, the load effect due to the slab is kept separate.

**Interior Strip (1'-0" Wide)**

Slab, wearing course, and barrier dead loads

$$W_{DC} = (1.0 \cdot 0.150 \cdot h) + \frac{2 \cdot (0.439)}{47.33} = 0.150 \cdot h + 0.019 \quad (\text{kip/ft})$$

Future wearing surface

$$W_{DW} = (1.0 \cdot 0.020) = 0.020 \quad (\text{kip/ft})$$

(included with DC loads in load tables)

**Exterior Strip (1'-0" Wide)**

Slab, wearing course, and barrier dead loads

$$W_{DC} = (1.0 \cdot 0.150 \cdot h) + \frac{0.439}{\left(\frac{69.4}{12}\right)} = 0.150 \cdot h + 0.076 \quad (\text{kip/ft})$$

Future wearing surface

$$W_{DW} = \frac{\left(\frac{69.4}{12} - 1.67\right) \cdot 0.020}{\left(\frac{69.4}{12}\right)} = 0.014 \quad (\text{kip/ft})$$

(included with DC loads in load tables)

### H. Summary of Analysis Results

From this point forward, the design of an interior strip (subject to dead and live loads) will be presented. The design procedure for the exterior strip is similar. A computer analysis was performed with a three-span continuous beam model. The model included the stiffening effect of the haunches.

Bending moment summaries obtained at different span locations are presented in Tables 5.7.1.1 through 5.7.1.4. These tables also contain truck live load deflections and dead load deflections due to slab selfweight, wearing course, and two barriers. Shear information is presented in Tables 5.7.1.5 through 5.7.1.7.

Loads and deflections that appear later in the example are identified with bold numbers.

**Table 5.7.1.1**  
**Moment Summary – One Lane**

Span Point	Lane (kip-ft)	Truck (kip-ft)	Tandem (kip-ft)	Truck Tr (kip-ft)	+ Fatigue (kip-ft)	- Fatigue (kip-ft)
1.0	0	0	0	-	0	0
1.1	50	194	178	-	140	-25
1.2	87	316	299	-	225	-49
1.3	112	373	368	-	285	-74
1.4	124	390	388	-	<b>299</b>	<b>-98</b>
1.5	124	374	374	-	285	-123
1.6	112/-76	333/-220	329/-178	-	253	-147
1.7	86/-87	244/-254	249/-204	-253	187	-172
1.8	-104	-289	-233	-292	128	-196
1.9	-149	-325	-263	-337	67	-258
2.0	-221	-378	-292	-383	<b>75</b>	<b>-387</b>
2.1	-129	-267	-229	-284	79	-228
2.2	44/-75	157/-226	190/-193	-226	151	-151
2.3	78/-64	284/-187	288/-163	-	223	-123
2.4	107	360	350	-	275	-95
2.5	117	378	368	-	284	-66

**Table 5.7.1.2**  
**Moment Summary – Interior Strip (per foot width)**

Span Point	M <sub>DC</sub> (kip-ft)	* Truck + Lane (kip-ft)	* Tandem + Lane (kip-ft)	* .9 (Truck Tr + Lane) (kip-ft)
1.0	0	0	0	-
1.1	17.2	25	23	-
1.2	28.9	41	39	-
1.3	34.3	49	48	-
1.4	<b>34.3</b>	51	51	-
1.5	28.9	50	50	-
1.6	17.1	44/-29	44/-25	-
1.7	-1.1	33/-34	34/-29	-31
1.8	-23.6	-39	-33	-35
1.9	-53.6	-47	-40	-43
2.0	<b>-90.9</b>	-58	-49	-53
2.1	-48.2	-39	-35	-37
2.2	-16.0	21/-30	24/-27	-27
2.3	7.5	37/-25	37/-22	-
2.4	20.4	47	46	-
2.5	25.7	50	49	-

\* Includes Dynamic Load Allowance (IM) and Live Load Distribution Factor.

**Table 5.7.1.3**  
**Moment Summary – Exterior Strip (per foot width)**

Span Point	M <sub>DC</sub> (kip-ft)	* Truck + Lane (kip-ft)	* Tandem + Lane (kip-ft)	* 0.9 (Truck Tr + Lane) (kip-ft)
1.0	0	0	0	-
1.1	20.0	32	30	-
1.2	33.3	53	51	-
1.3	40.6	63	62	-
1.4	40.6	67	67	-
1.5	33.3	64	64	-
1.6	19.8	58/-38	57/-33	-
1.7	-1.0	44/-44	45/-37	-40
1.8	-28.1	-51	-43	-46
1.9	-62.5	-60	-52	-56
2.0	-105.2	-75	-63	-69
2.1	-55.2	-50	-45	-48
2.2	-18.7	28/-40	31/-34	-35
2.3	8.3	48/-33	48/-29	-
2.4	24.0	61	59	-
2.5	29.2	64	63	-

\*Includes Dynamic Load Allowance (IM) and Live Load Distribution Factor.

**Table 5.7.1.4**  
**Moment Load Combinations and Deflections**

Span Point	Service I		Strength I		* Lane LL Deflection (in)	* Truck LL Deflection (in)	** Dead Ld Deflection (in)
	Interior (kip-ft)/ft	Exterior (kip-ft)/ft	Interior (kip-ft)/ft	Exterior (kip-ft)/ft			
1.0	0	0	0	0	0.000	0.000	0.000
1.1	42	52	65	81	0.047	0.172	0.089
1.2	70	86	107	134	0.087	0.310	0.162
1.3	83	104	128	162	0.118	0.414	0.209
1.4	<b>86</b>	107	<b>133</b>	167	0.137	0.466	0.224
1.5	79	98	123	154	<b>0.141</b>	<b>.0475</b>	0.208
1.6	<b>62/-12</b>	78/-18	<b>99/-35</b>	127/-49	0.131	0.430	0.166
1.7	<b>32/-35</b>	42/-45	<b>58/-61</b>	77/-78	0.108	0.344	0.110
1.8	<b>-63</b>	-79	<b>-98</b>	-124	0.076	0.242	0.056
1.9	<b>-100</b>	-123	<b>-148</b>	-184	0.038	0.120	0.019
2.0	<b>-149</b>	-180	<b>-215</b>	-263	0.000	0.000	0.000
2.1	<b>-87</b>	-105	<b>-128</b>	-156	0.046	0.156	0.002
2.2	<b>8/-46</b>	13/-58	<b>28/-73</b>	37/-93	0.072	0.328	0.031
2.3	<b>44/-18</b>	56/-25	<b>74/-37</b>	94/-50	0.138	0.500	0.085
2.4	67	85	108	137	0.167	0.586	0.130
2.5	75	94	<b>119</b>	149	<b>0.178</b>	<b>0.653</b>	0.147

\* Based on  $I_{\text{effective}} = \frac{1}{2} I_{\text{gross}}$ . Includes LL distribution factor.

\*\*Includes selfweight, wearing course, and barriers.

**Table 5.7.1.5**  
**Shear Summary – One Lane**

Span Point	Lane (kips)	Truck (kips)	Tandem (kips)
1.0	12.7	52.8	47.0
1.1	10.3	44.1	40.4
1.2	7.8	35.8	34.0
1.3	5.9	28.2	27.8
1.4	5.8	21.1	22.8
1.5	7.3	27.6	28.5
1.6	9.2	34.9	33.9
1.7	11.4	42.3	38.6
1.8	13.9	49.3	42.6
1.9	16.4	55.6	46.0
2.0	19.9	61.2	48.8
2.1	16.6	54.4	45.0
2.2	13.4	46.9	40.4
2.3	10.6	38.8	35.0
2.4	8.2	30.5	29.0
2.5	6.2	22.4	22.7

**Table 5.7.1.6**  
**Shear Summary (per foot width)**

Span Point	V <sub>DC</sub> (kips)	* Truck + Lane (kips)	* Tandem + Lane (kips)
1.0	4.6	4.5	4.1
1.1	3.3	3.7	3.5
1.2	2.0	3.0	2.9
1.3	0.7	2.3	2.3
1.4	0.7	1.8	2.0
1.5	2.0	2.4	2.4
1.6	3.3	3.0	2.9
1.7	4.7	3.7	3.4
1.8	6.0	4.3	3.8
1.9	7.5	4.9	4.2
2.0	9.5	5.5	4.6
2.1	6.7	4.8	4.1
2.2	5.0	4.1	3.6
2.3	3.3	3.4	3.1
2.4	1.7	2.6	2.5
2.5	0.0	1.9	2.0

\* Includes Dynamic Load Allowance (IM) and 0.054 Distribution Factor.

**Table 5.7.1.7**  
**Shear Summary – Load Combinations**

Span Point	SERVICE I (kips)	STRENGTH I (kips)
1.0	9.1	13.6
1.1	7.0	10.6
1.2	5.0	7.7
1.3	3.0	5.0
1.4	2.7	4.3
1.5	4.4	6.8
1.6	6.3	9.4
1.7	8.4	12.3
1.8	10.3	<b>15.0</b>
1.9	12.4	<b>17.9</b>
2.0	15.0	21.4
2.1	11.5	16.8
2.2	9.1	13.4
2.3	6.7	10.0
2.4	4.3	6.7
2.5	2.0	3.4

**I. Live Load  
Deflection  
[2.5.2.6]**

To prevent serviceability problems, a limit is placed on the maximum live load deflections. The limit is:

$$\Delta_{LL+I} = \frac{\text{Span}}{800}$$

$$\text{Spans 1 and 3} = \frac{44 \cdot 12}{800} = 0.66 \text{ in}$$

$$\text{Span 2} = \frac{55 \cdot 12}{800} = 0.83 \text{ in}$$

**[3.6.1.3.2]**

Use the design truck alone or design lane load plus 25% of truck load.

Using the Table 5.7.1.1 live load deflection values, the following maximum live load deflections were obtained:

Midspans 1 and 3

$$\text{Truck:} \quad 0.475 \text{ in} < 0.66 \text{ in}$$

$$\text{Lane} + 25\% \text{ Truck: } 0.141 + 0.25 (0.475) = 0.260 \text{ in} < 0.66 \text{ in}$$

Midspan 2

$$\text{Truck:} \quad 0.653 \text{ in} < 0.83 \text{ in}$$

$$\text{Lane} + 25\% \text{ Truck: } 0.178 + 0.25 (0.653) = 0.341 \text{ in} < 0.66 \text{ in}$$

**J. Shear in Slab  
[5.13.3.6]**

$$V_r = \phi \cdot V_n = 0.9 V_n$$

Check the one-way shear capacity of the slab.

**[5.8.2.9]**

**Critical Section**

Shear should be checked at all sections. In many cases the governing location is at the abutment, a pier, or at the start of the haunch. Calculations for the shear check at the start of the linear haunch for the side span (Span Point 1.81) follow.

The effective shear depth  $d_v$  is the distance between the internal tension and compression force components to resist flexure, which is unknown at this point in the design.

But the shear depth need not be less than

$$0.9 \cdot d_e = 0.9 \cdot (17.0) = 15.30 \text{ in}$$

or

$$0.72 \cdot h = 0.72 \cdot (19.0) = 13.68 \text{ in}$$

Use  $d_v = 15.30 \text{ in}$

The shear loads at adjacent span points are interpolated to determine the shear at Span Point 1.81:

$$V_U = 15.0 \text{ kips} + \left( \frac{1.81 - 1.8}{1.9 - 1.8} \right) \cdot (17.9 \text{ kips} - 15.0 \text{ kips}) = 15.3 \text{ kips}$$

**[5.8.3.3]**

**Nominal Shear Resistance**

The nominal shear resistance is the sum of the contributions from the concrete and steel.

$$V_n = V_c + V_s$$

It can be no more than:

$$V_n \leq 0.25 \cdot f'_c \cdot b_v \cdot d_v = 0.25 \cdot 4.0 \cdot 12 \cdot 15.30 = 183.6 \text{ kips}$$

To simplify the calculation for the concrete contribution, assume  $\beta = 2.0$ . If shear reinforcement is found necessary, the designer should first try thickening the slab to eliminate the need for shear reinforcement. If shear reinforcement must be used, the appropriate  $\beta$  and  $\theta$  values should be used for the shear design.

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 2.0 \cdot \sqrt{4.0} \cdot 12 \cdot 15.30 = 23.2 \text{ kips}$$

Without shear reinforcement,  $V_s = 0$

The nominal shear capacity of the slab is:

$$V_n = 23.2 + 0 = 23.2 \text{ kips} < 183.6 \text{ kips} \quad \underline{\text{OK}}$$

Check if the shear resistance is greater than the shear demand:

$$V_r = \phi \cdot V_n = 0.90 \cdot (23.2) = 20.9 \text{ kips} > 15.3 \text{ kips} \quad \underline{\text{OK}}$$

**K. Design Positive Moment Reinforcement**

[5.7.2.2]  
[5.7.3.2]

Determine the required area of flexural reinforcement to satisfy the Strength I Load Combination.

**Flexural Resistance**

Assume a rectangular stress distribution and solve for the required area of reinforcing based on  $M_u$  and  $d$ . Also assume a resistance factor of 0.9.

For  $f'_c = 4.0$  ksi,  $\beta_1 = 0.85$

$$M_u = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left( d - \frac{a}{2} \right)$$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

$$M_u = \phi \cdot A_s \cdot f_y \cdot \left( d - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b} \right)$$

$$M_u = 0.90 \cdot A_s \cdot (60) \cdot \left( d - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 12} \right) \cdot \left( \frac{1}{12} \right)$$

$$3.309 \cdot A_s^2 - 4.5 \cdot d \cdot A_s + M_u = 0$$

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618}$$

The "d" value used in positive moment regions does not include the 2 inch wearing course.

$$d_{int} = 21 - 2 - 1.5 - 0.5 \cdot (1.0) = 17.00 \text{ in}$$

$$d_{ext} = 21 - 2 - 1.5 - 0.5 \cdot (1.27) = 16.86 \text{ in}$$

Trial reinforcement information for Span Points 1.4 and 2.5 are provided in the following table. After evaluating the areas of steel required, a layout based on a 5 inch base dimension was selected for the interior strip.

**Trial Bottom Longitudinal Reinforcement**

Span Point	Interior Strip					Exterior Strip				
	$M_u$	d	$A_s$ (req)	Trial Bars	$A_s$ (prov)	$M_u$	d	$A_s$ (req)	Trial Bars	$A_s$ (prov)
1.4	133	17.00	1.89	#8 @ 5	1.90	167	16.86	2.47	#10 @ 6	2.54
2.5	119	17.00	1.68	#8 @ 5	1.90	149	16.86	2.17	#10 @ 6	2.54

**[5.5.4.2.1]**

Validate the assumption of 0.9 for resistance factor:

Calculate the depth of the Whitney stress block.

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{1.90 \cdot 60}{0.85 \cdot 4 \cdot 12} = 2.79$$

The depth of the section in compression is:

$$c = \frac{a}{\beta_1} = \frac{2.79}{0.85} = 3.28$$

$$\phi = 0.65 + 0.15 \cdot \left( \frac{d_t}{c} - 1 \right) = 0.65 + 0.15 \cdot \left( \frac{17.00}{3.28} - 1 \right) = 1.28 > 0.9$$

Therefore,  $\phi = 0.9$

**[5.7.3.4]****Crack Control**

To ensure that cracking is limited to small cracks that are well distributed, a limit is placed on the spacing of the reinforcing steel. LRFD Equation 5.7.3.4-1 defines the maximum spacing permitted:

$$s \leq s_{\max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

At Span Point 1.4 the Service I positive moment is 86 kip-ft.

The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. To simplify the calculations, the section is assumed to be singly reinforced.

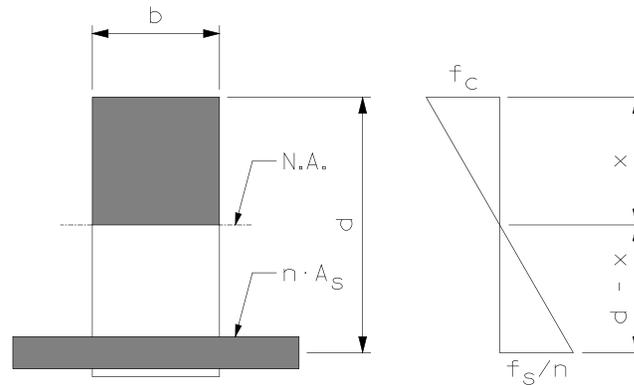
**[5.4.2.4 & 5.7.1]**

$$n = \frac{E_s}{E_c} = \frac{29,000}{33,000 \cdot (0.145)^{1.5} \cdot \sqrt{4.0}} = 7.96$$

Use  $n = 8$

$$n \cdot A_s = 8 \cdot (1.90) = 15.2 \text{ in}^2$$

Determine the location of the neutral axis:



$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)$$

$$\frac{(12) \cdot x^2}{2} = 15.2 \cdot (17.0 - x) \quad \text{solving, } x = 5.42 \text{ in}$$

Determine the lever arm between service load flexural force components.

$$j \cdot d = d - \frac{x}{3} = 17.0 - \frac{5.42}{3} = 15.2 \text{ in}$$

Compute the stress in the reinforcement.

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{86 \cdot 12}{1.90 \cdot (15.2)} = 35.7 \text{ ksi}$$

For  $d_c = 2.00$  in (1.5 in +  $\frac{1}{2}$  of #25 bar)

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2}{0.7 \cdot (21 - 2)} = 1.15$$

Use  $\gamma_e = 0.75$

$$s_{\max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.15 \cdot 35.7} - 2 \cdot 2 = 8.8 \text{ in} > 5 \text{ in} \quad \text{OK}$$

### [5.5.3]

#### Fatigue

The stress range in the reinforcement is computed and compared against code limits to ensure adequate fatigue resistance is provided.

### [Table 3.4.1-1]

$U = 1.50$  (LL + IM)

### [3.6.2.1]

IM = 15%

At Span Point 1.4 the one lane fatigue moments are:

Maximum positive moment = 299 kip-ft

Maximum negative moment = -98 kip-ft

Multiplying the one lane moments by the appropriate load factor, dynamic load allowance, and distribution factor results in the following fatigue moments:

$$\text{Fatigue LL } M_{\max} = 299 \cdot (1.50) \cdot 1.15 \cdot 0.052 = 26.8 \text{ kip-ft}$$

$$\text{Fatigue LL } M_{\min} = -98 \cdot (1.50) \cdot 1.15 \cdot 0.052 = -8.8 \text{ kip-ft}$$

The unfactored dead load moment at Span Point 1.4 is 34.3 kip-ft.

The moments on the cross section when fatigue loading is applied are:

Maximum moment = 34.3 + 26.8 = 61.1 kip-ft

Minimum moment = 34.3 - 8.8 = 25.5 kip-ft

Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:

For the maximum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{61.1 \cdot 12}{1.90 \cdot (15.2)} = 25.4 \text{ ksi}$$

For the minimum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{25.5 \cdot 12}{1.90 \cdot (15.2)} = 10.6 \text{ ksi}$$

The stress range in the reinforcement ( $f_f$ ) is the difference between the two stresses

$$f_f = (25.4 - 10.6) = 14.8 \text{ ksi}$$

**[5.5.3.2]**

The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.

$$\begin{aligned} f_{f(\max)} &= 24 - 0.33 \cdot f_{\min} = 24 - 0.33 \cdot (10.6) \\ &= 20.5 > 14.8 \text{ ksi} \quad \text{OK} \end{aligned}$$

**[5.7.3.3.2]****Check Minimum Reinforcement**

To prevent a brittle failure, adequate flexural reinforcement needs to be placed in the cross section. For this check, the thickness of the slab including the wearing course is used to be conservative.

$$f_r = 0.37 \cdot \sqrt{f'_c} = 0.37 \cdot \sqrt{4} = 0.74 \text{ ksi}$$

$$t = 21.0 \text{ in}$$

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (21)^3 = 9261 \text{ in}^4$$

$$y_t = 10.5 \text{ in}$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.74 \cdot 9261}{10.5 \cdot (12)} = 54.4 \text{ kip-ft}$$

$$1.2 M_{cr} = 65.3 \text{ kip-ft}$$

$$M_r = \phi A_s f_y \cdot \left( d - \frac{a}{2} \right)$$

$$M_r = 0.9 \cdot (1.90) \cdot (60) \cdot \left( 17.0 - \frac{2.79}{2} \right) \cdot \frac{1}{12}$$

$$M_r = 133.4 \text{ kip-ft} > 1.2 M_{cr} = 65.3 \text{ kip-ft}$$

OK

Use #8 bars at 5 inches at Span Point 1.4

**[5.11.1.2.1]****[5.11.1.2.2]****Bar Cutoff Location**

Determine the location where the 5 inch spacing can be increased to 10 inches. Assume that the bars will be dropped in non-haunched regions of the span. The moment capacity of #8 bars at 10 inches ( $A_s = 0.95 \text{ in}^2$ ) for positive flexure is:

$$M_r = \phi A_s f_y \cdot \left( d - \frac{a}{2} \right)$$

$$M_r = 0.9 \cdot (0.95) \cdot (60) \cdot \left( 17.0 - \frac{0.95 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)} \right) \cdot \frac{1}{12} = 69.7 \text{ kip-ft}$$

For the interior strip, the positive bending moments are:

Span Point	M <sub>Strength I</sub> (kip-ft)/ft	M <sub>Service I</sub> (kip-ft)/ft
1.6	99	62
1.7	58	32
2.2	28	8
2.3	74	44

Knowing that span points are 4.4 feet apart in Span 1 and 5.5 feet apart in Span 2, the drop point locations which meet the positive Strength I bending moment of 69.7 kip-ft can be found.

For Span 1, interpolate between Span Points 1.6 and 1.7:

$$1.6 + \left( \frac{99 - 69.7}{99 - 58} \right) \cdot 0.1 = 1.67 \text{ or } 14.5 \text{ ft from Pier 1 centerline.}$$

For Span 2, interpolate between Span Points 2.2 and 2.3:

$$2.2 + \left( \frac{69.7 - 28}{74 - 28} \right) \cdot 0.1 = 2.29 \text{ or } 16.0 \text{ ft from Pier 1 centerline.}$$

The reinforcement must also meet the serviceability requirements at the theoretical drop point. Determine the drop point location based on the crack control requirements and compare with the drop points based on strength to see which ones govern.

#### [5.7.3.4]

For #8 bars @ 10", ( $A_s = 0.95 \text{ in}^2$ ), and  $d_c = 2 \text{ in}$  :

$$s = 10 \text{ in} \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

where

$$f_{ss} = \frac{M_{\text{drop}}}{A_s \cdot j \cdot d}$$

Determine neutral axis:

$$n \cdot A_s = 8 \cdot 0.95 = 7.60 \text{ in}^2$$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)$$

$$\frac{12 \cdot x^2}{2} = 7.60 \cdot (17.00 - x) \quad \text{solving, } x=4.05 \text{ in}$$

$$\text{Then } j \cdot d = d - \frac{x}{3} = 17.00 - \frac{4.05}{3} = 15.65 \text{ in}$$

$$s = 10 \text{ in} \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot \frac{M_{\text{drop}}}{A_s \cdot j \cdot d}} - 2 \cdot d_c$$

Solving for  $M_{\text{drop}}$ :

$$M_{\text{drop}} = \frac{700 \cdot \gamma_e \cdot A_s \cdot j \cdot d}{\beta_s \cdot (s + 2 \cdot d_c)} = \frac{700 \cdot 0.75 \cdot 0.95 \cdot 15.65}{1.15 \cdot (10 + 2 \cdot 2)} \cdot \frac{1}{12} = 40.4 \text{ kip-ft}$$

Interpolate to determine span point location of drop point:

For Span 1:

$$1.6 + \left( \frac{62 - 40.4}{62 - 32} \right) \cdot 0.1 = 1.67 \text{ or } 14.5 \text{ ft from Pier 1 centerline.}$$

For Span 2:

$$2.2 + \left( \frac{40.4 - 8}{44 - 8} \right) \cdot 0.1 = 2.29 \text{ or } 16.0 \text{ ft from Pier 1 centerline.}$$

Therefore, the drop point locations based on crack control match those based on strength.

By inspection, the fatigue stress range check and the minimum reinforcement check are satisfied.

#### [5.11.1.2.1]

Due to the uncertainty associated with the design moments, the reinforcement cannot be terminated at the theoretical drop point. It must be carried beyond the theoretical point by the greater of: the depth of the member, 15 times the nominal diameter of the bar, or  $1/20$  of the clear span.

The required extension  $L_{ext1}$  for Span 1 is:

$$L_{ext1} = d = 17.0 \text{ in}$$

or

$$L_{ext1} = 15 \cdot d_b = 15 \cdot 1.00 = 15.0 \text{ in}$$

or

$$L_{ext1} = \frac{1}{20} \cdot (44 \cdot 12) = 26.4 \text{ in} \quad \underline{\text{GOVERNS}}$$

The required extension  $L_{ext2}$  for Span 2 is:

$$L_{ext2} = \frac{1}{20} \cdot (55 \cdot 12) = 33.0 \text{ in}$$

Adding the extension length to the theoretical distance from the pier at which the bars can be dropped results in the following cutoff locations:

$$\text{For Span 1: } 14.5 - \frac{26.4}{12} = 12.3 \text{ ft} \quad \underline{\text{Use 12'-0"}}$$

$$\text{For Span 2: } 16.0 - \frac{33.0}{12} = 13.25 \text{ ft} \quad \underline{\text{Use 13'-0"}}$$

By continuing half of the reinforcement for the entire length of the bridge, LRFD Article 5.11.1.2.2 is satisfied.

### **[5.8.3.5]**

#### **Check Longitudinal Reinforcement**

Check the minimum longitudinal reinforcement requirements at the abutments, assuming that a diagonal crack would start at the inside edge of the bearing area.

The slab sits on a 2'-10" wide integral abutment.

For  $\theta = 45^\circ$  determine the length from the end of the slab,  $L_{crack}$ , at which a diagonal crack will intersect the bottom longitudinal reinforcement (#8 bars @ 5"):

$$L_{crack} = 2.83 + \left( \frac{2.00}{12} \right) \cot(45^\circ) = 3.00 \text{ ft} = 36.00 \text{ in}$$

From Figure 5.2.2.2 of this manual, the development length for #8 bars @ 5" with 1.5" cover is:

$$\ell_{d25} = 3'-9" = 45"$$

Then the tensile resistance of the longitudinal bars at the crack location

$$\begin{aligned} T_r &= f_y \cdot A_s = \frac{L_c - (\text{end cover})}{\ell_{d25}} \\ &= 60 \cdot 1.90 \cdot \left( \frac{36.0 - (\sim 3.5)}{45.0} \right) = 82.3 \text{ kips} \end{aligned}$$

The force to be resisted is:

$$\begin{aligned} T &= \left( \frac{V_u}{\phi} - 0.5V_s - V_p \right) \cot \theta \\ &= \left( \frac{13.6}{0.9} - 0.5 \cdot 0 - 0 \right) \cot 45^\circ \\ &= 15.1 \text{ kips} < 82.3 \text{ kips} \quad \text{OK} \end{aligned}$$

Note that LRFD C5.8.3.5 states that  $V_u$  may be taken at  $0.5d_v \cot \theta$  or  $d_v$  away from the face of support. For simplicity, the value for  $V_u$  at the abutment centerline of bearing was used in the equation above.

**L. Design Negative  
Moment  
Reinforcement**  
[5.7.2.2]  
[5.7.3.2]

Determine the required area of flexural reinforcement to satisfy the Strength I Load Combination.

**Flexural Resistance**

Assume a rectangular stress distribution and solve for the required area of reinforcing based on  $M_u$  and  $d$ .

Use the same general equation developed for the positive moment reinforcement.

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.23 \cdot M_u}}{6.618}$$

$$d_{int} = 36 - 3 - 0.5 \cdot (1.0) = 32.50 \text{ in}$$

$$d_{ext} = 36 - 3 - 0.5 \cdot (1.128) = 32.44 \text{ in}$$

The required area of steel and trial reinforcement is presented in the following table.

**Trial Top Longitudinal Reinforcement**

Span Point	Interior Strip					Exterior Strip				
	M <sub>U</sub>	d	A <sub>S</sub> (req)	Trial Bars	A <sub>S</sub> (prov)	M <sub>U</sub>	d	A <sub>S</sub> (req)	Trial Bars	A <sub>S</sub> (prov)
2.0	-215	32.50	1.52	#8 @ 5	1.90	-263	32.44	1.88	#8 @ 6	2.00

By inspection, the resistance factor is equal to 0.9

**[5.7.3.4]**

**Crack Control**

At Span Point 2.0 the Service I moment is -149 kip-ft

Similar to the positive moment sections, the stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. For this check, the section is assumed to be singly reinforced.

$$n \cdot A_s = 8 \cdot (1.90) = 15.2 \text{ in}^2$$

Determine the location of the neutral axis:

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)$$

$$\frac{(12) \cdot x^2}{2} = 15.2 \cdot (32.5 - x) \quad \text{solving, } x=7.90 \text{ in}$$

Determine the lever arm between service load flexural force components.

$$j \cdot d = d - \frac{x}{3} = 32.5 - \frac{7.90}{3} = 29.9 \text{ in}$$

Compute the stress in the reinforcement.

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{149 \cdot 12}{1.90 \cdot (29.9)} = 31.5 \text{ ksi}$$

For  $d_c = 2.50$  in (2.0 in max cover +  $\frac{1}{2}$  of #8 bar)

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.5}{0.7 \cdot (36 - 2.5)} = 1.11$$

Use  $\gamma_e = 0.75$ :

$$s \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.11 \cdot 31.5} - 2 \cdot 2.5 = 10.0 \text{ in} > 5 \text{ in } \underline{\text{OK}}$$

### [5.5.3]

#### Fatigue

The stress range in the reinforcement is computed and compared against code limits to ensure adequate fatigue resistance is provided.

### [Table 3.4.1-1]

$U = 1.5$  (LL + IM)

IM = 15%

### [3.6.2.1]

At Span Point 2.0 the one lane fatigue moments are:

Maximum positive moment = 75 kip-ft

Maximum negative moment = -387 kip-ft

Multiplying the one lane moments by the appropriate load factor, dynamic load allowance, and distribution factor results in the following fatigue moments:

$$\text{Fatigue LL } M_{\max} = 75 \cdot (1.50) \cdot 1.15 \cdot 0.052 = 6.7 \text{ kip-ft}$$

$$\text{Fatigue LL } M_{\min} = -387 \cdot (1.50) \cdot 1.15 \cdot 0.052 = -34.7 \text{ kip-ft}$$

The unfactored dead load moment at Span Point 2.0 is -90.9 kip-ft.

The moments on the cross section when fatigue loading is applied are:

$$\text{Maximum moment} = -90.9 + 6.7 = -84.2 \text{ kip-ft}$$

$$\text{Minimum moment} = -90.9 - 34.7 = -125.6 \text{ kip-ft}$$

Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:

For the maximum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{84.2 \cdot 12}{1.90 \cdot (29.9)} = 17.8 \text{ ksi}$$

For the minimum moment:

$$f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{125.6 \cdot 12}{1.90 \cdot (29.9)} = 26.5 \text{ ksi}$$

The stress range in the reinforcement ( $f_f$ ) is the difference between the two stresses.

$$f_f = (26.5 - 17.8) = 8.7 \text{ ksi}$$

**[5.5.3.2]**

The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.

$$\begin{aligned} f_{f(\max)} &= 24 - 0.33 \cdot f_{\min} = 24 - 0.33 \cdot (17.8) \\ &= 18.1 \text{ ksi} > 8.7 \text{ ksi} \quad \text{OK} \end{aligned}$$

**[5.7.3.3.2]****Check Minimum Reinforcement**

To prevent a brittle failure, adequate flexural reinforcement needs to be placed in the cross section.

$$f_r = 0.74 \text{ ksi}$$

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (36)^3 = 46,656 \text{ in}^4$$

$$y_t = 18.0 \text{ in}$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.74 \cdot 46,656}{18.0 \cdot (12)} = 159.8 \text{ kip-ft}$$

$$1.2 M_{cr} = 191.8 \text{ kip-ft}$$

$$M_r = \phi A_s f_y \cdot \left( d - \frac{a}{2} \right)$$

$$M_r = 0.9 \cdot (1.90) \cdot (60) \cdot \left( 32.5 - \frac{2.79}{2} \right) \cdot \frac{1}{12}$$

$$= 265.9 \text{ kip-ft} > 1.2 M_{cr} = 191.8 \text{ kip-ft} \quad \text{OK}$$

Use #8 bars at 5 inches at Span Point 2.0

**[5.11.1.2.1]****[5.11.1.2.3]****Bar Cutoff Location**

Determine the location where the 5 inch spacing can be increased to 10 inches. Assume that the bars will be dropped in non-haunched regions of the span. The moment capacity of #8 bars at 10 inches ( $A_s = 0.95 \text{ in}^2$ ) for negative flexure is:

$$M_r = \phi A_s f_y \cdot \left( d - \frac{a}{2} \right)$$

$$= 0.9 \cdot (0.95) \cdot (60) \cdot \left[ 17.5 - \frac{0.95 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)} \right] \cdot \frac{1}{12} = 71.8 \text{ kip-ft}$$

For the interior strip, the negative bending moments are:

Span Point	M <sub>Strength I</sub> (kip-ft)/ft	M <sub>Service I</sub> (kip-ft)/ft
1.6	-35	-12
1.7	-61	-35
1.8	-98	-63
1.9	-148	-100
2.0	-215	-149
2.1	-128	-87
2.2	-73	-46
2.3	-37	-18

Knowing that span points are 4.4 feet apart in Span 1 and 5.5 feet apart in Span 2, the drop point locations which meet the Strength I negative bending moment of 71.8 kip-ft can be found.

For Span 1, interpolate between Span Points 1.7 and 1.8:

$$1.7 + \left( \frac{71.8 - 61}{98 - 61} \right) \cdot 0.1 = 1.73 \text{ or } 11.9 \text{ ft from Pier 1 centerline.}$$

For Span 2, interpolate between Span Points 2.2 and 2.3:

$$2.2 + \left( \frac{73 - 71.8}{73 - 37} \right) \cdot 0.1 = 2.20 \text{ or } 11.0 \text{ ft from Pier 1 centerline.}$$

The reinforcement must also meet the serviceability requirements at the theoretical drop point. Determine the drop point location based on the crack control requirements and compare with the drop points based on strength to see which ones govern.

**[5.7.3.4]**

For #8 bars @ 10", ( $A_s = 0.95 \text{ in}^2$ ),  $d_c = 2.50 \text{ in}$ :

Determine neutral axis:

$$n \cdot A_s = 8 \cdot 0.95 = 7.60 \text{ in}^2$$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)$$

$$\frac{12 \cdot x^2}{2} = 7.60 \cdot (17.5 - x) \quad \text{solving, } x = 4.12 \text{ in}$$

$$\text{Then } j \cdot d = d - \frac{x}{3} = 17.5 - \frac{4.12}{3} = 16.13 \text{ in}$$

$$\beta_s = 1 + \frac{2.50}{0.7(21 - 2.50)} = 1.19$$

Solve for the moment at the drop point:

$$M_{\text{drop}} = \frac{700 \cdot \gamma_c \cdot A_s \cdot j \cdot d}{\beta_s \cdot (s + 2 \cdot d_c)} = \frac{700 \cdot 0.75 \cdot 0.95 \cdot 16.13}{1.19 \cdot (10 + 2 \cdot 2.5)} \cdot \frac{1}{12} = 37.6 \text{ kip-ft}$$

Interpolate to determine span point location of drop point:

For Span 1:

$$1.7 + \left( \frac{37.6 - 35}{63 - 35} \right) \cdot 0.1 = 1.71 \text{ or } 12.8 \text{ ft from Pier 1 centerline.}$$

For Span 2:

$$2.2 + \left( \frac{46 - 37.6}{46 - 18} \right) \cdot 0.1 = 2.23 \text{ or } 12.7 \text{ ft from Pier 1 centerline}$$

Therefore, the drop point locations based on crack control govern the bar cutoff locations.

By inspection, the fatigue stress range check and the minimum reinforcement check are satisfied.

**[5.11.1.2.1]**

Due to the uncertainty associated with the design moments, the reinforcement cannot be terminated at the theoretical drop point. It must be carried beyond the theoretical point by the greater of: the depth of the member, 15 times the nominal diameter of the bar, or  $1/20$  of the clear span.

The required extension  $L_{ext1}$  for Span 1 is:

$$L_{ext1} = d = 21 - 3 - 0.5 \cdot (1.0) = 17.5 \text{ in}$$

or

$$L_{ext1} = 15 \cdot d_b = 15 \cdot 1.00 = 15.0 \text{ in}$$

or

$$L_{ext1} = \frac{1}{20} \cdot (44 \cdot 12) = 26.4 \text{ in} \quad \text{GOVERNS}$$

The required extension  $L_{ext2}$  for Span 2 is:

$$L_{ext2} = \frac{1}{20} \cdot (55 \cdot 12) = 33.0 \text{ in}$$

Adding the extension length to the theoretical distance from the pier at which the bars can be dropped results in the following cutoff locations from the pier:

$$\text{For Span 1: } 12.8 + \frac{26.4}{12} = 15.0 \text{ ft} \quad \text{Use 15'-0"}$$

$$\text{For Span 2: } 12.7 + \frac{33.0}{12} = 15.5 \text{ ft} \quad \text{Use 15'-6"}$$

By continuing half of the reinforcement for the entire length of the bridge, LRFD Article 5.11.1.2.3 is satisfied.

**M. Distribution  
Reinforcement  
[5.14.4.1]**

The amount of transverse reinforcement may be taken as a percentage of the main reinforcement required:

$$\frac{100}{\sqrt{L}} \leq 50\%$$

$$\frac{100}{\sqrt{44}} = 15.1\%$$

For the interior strip, the maximum reinforcement is #8 bars at 5 inches (1.90 in<sup>2</sup>/ft). The required transverse reinforcement for load distribution is:

$$0.151 \cdot (1.90) = 0.29 \text{ in}^2/\text{ft}$$

Use #5 @ 12",  $A_s = 0.31 \text{ in}^2/\text{ft}$  for bottom transverse reinforcement.

**N. Shrinkage and Temperature Reinforcement**  
[5.10.8]

Adequate reinforcement needs to be provided in the slab to ensure that cracks from shrinkage and temperature changes are small and well distributed.

$$\text{Temperature } A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.3 \cdot 568 \cdot 21}{2 \cdot (568 + 21) \cdot 60} = 0.22 \text{ in}^2/\text{ft}$$

(total in each direction, on each face)

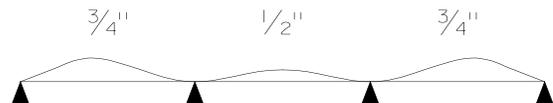
$$0.11 \leq A_s \leq .60 \quad \text{use } A_s = 0.22 \text{ in}^2/\text{ft}$$

Use #5 @ 12",  $A_s = 0.31 \text{ in}^2/\text{ft}$  for transverse top reinforcement.

**O. Dead Load Camber**  
[5.7.3.6.2]

The total weight of the superstructure is used for dead load deflections. The gross moment of inertia is used and a computer analysis is used to obtain instantaneous deflections. A longtime deflection multiplier of 4.0 is used in conjunction with the gross moment of inertia. The slab is cambered upward an amount equal to the immediate deflection + 1/2 of the long-term deflection. A camber diagram for the interior strip is shown below:

Instantaneous + 1/2 Long-term Camber



**P. Final Reinforcement Layout**

Figure 5.7.1.3 contains a plan view and Figure 5.7.1.4 contains a cross section that illustrates the reinforcement for the slab. As one would expect, the figures show that the exterior strips contain more reinforcing steel than the interior of the slab.

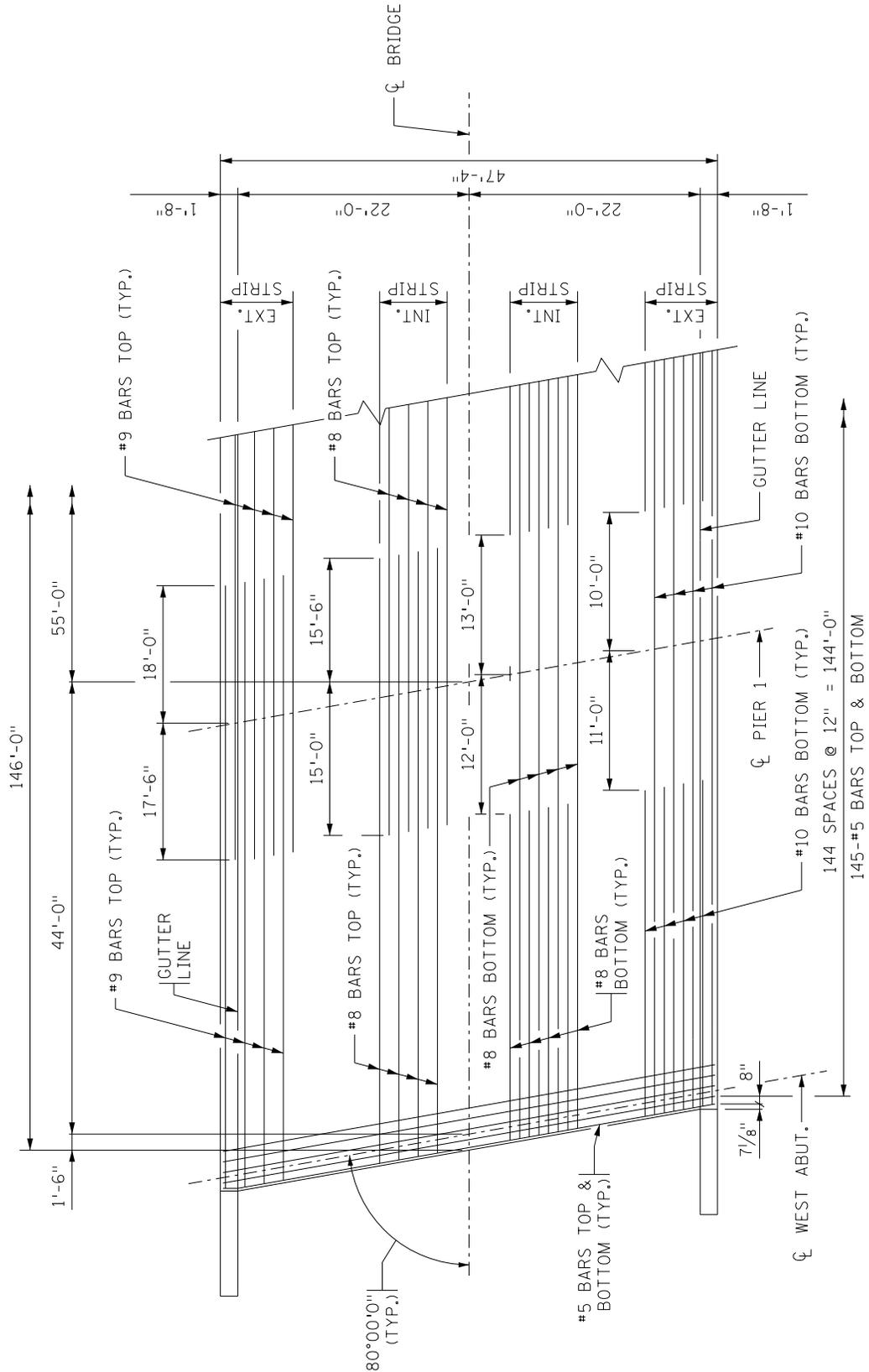


Figure 5.7.1.3

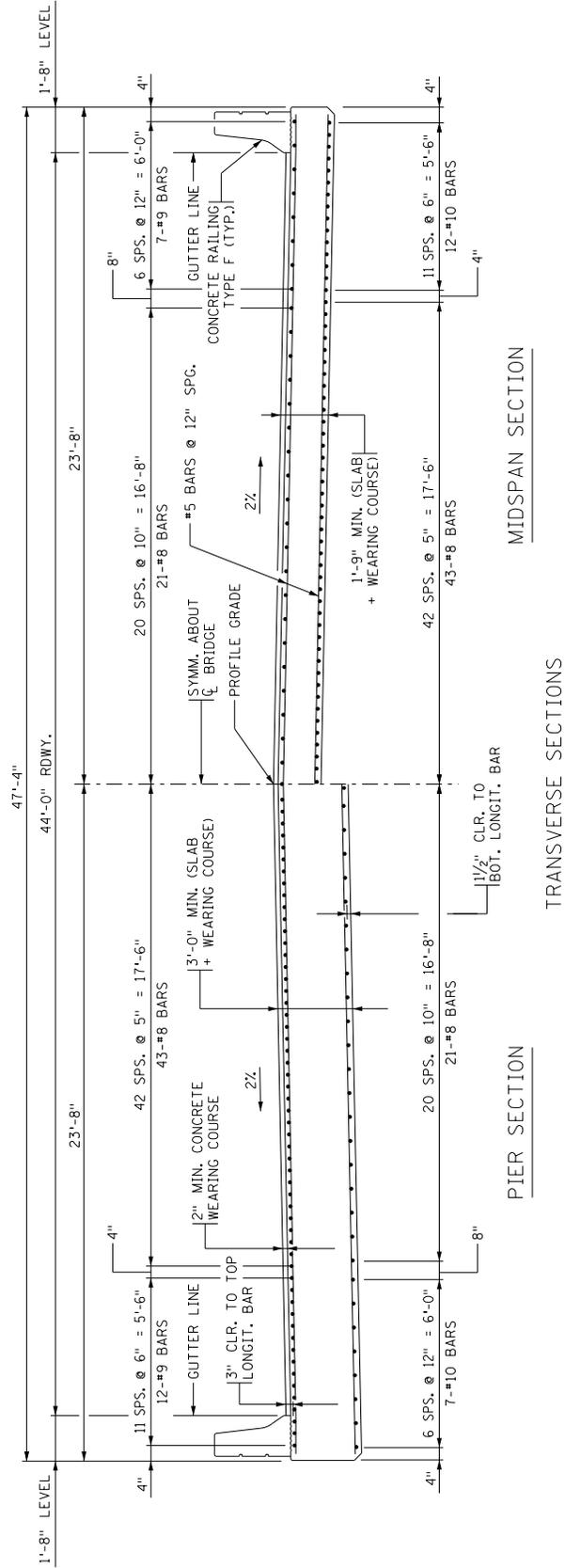
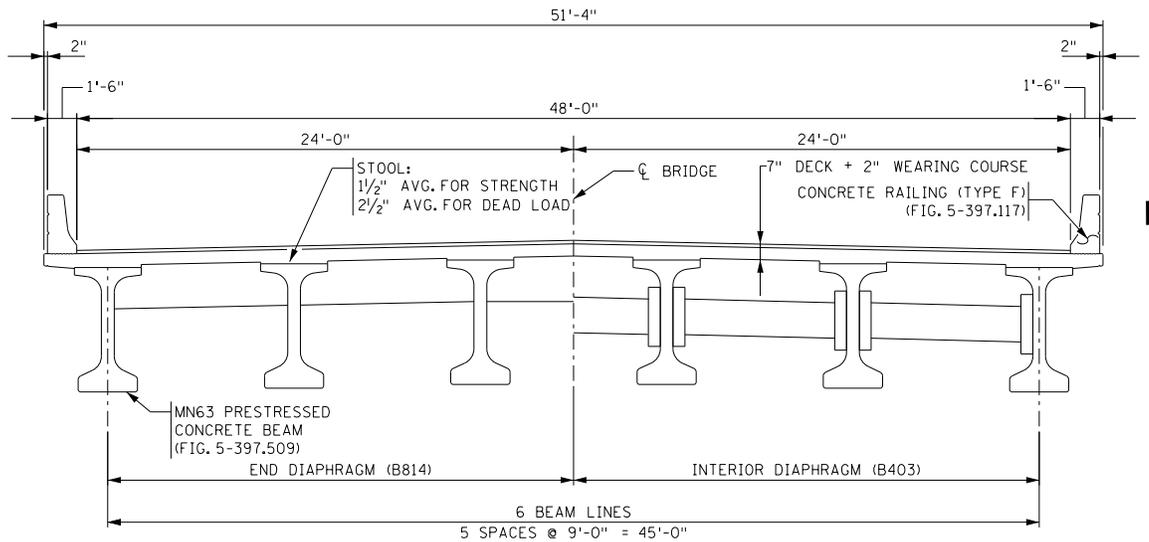


Figure 5.7.1.4

**5.7.2 Prestressed  
I-Beam Design  
Example**

This example illustrates the design of a pretensioned I-Beam for a two span bridge without skew. The 137'-0" spans are supported with MnDOT "MN63" beams. MnDOT standard details and drawings for diaphragms (B403, B814), railings (Fig. 5-397.117), and beams (Fig. 5-397.509) are to be used with this example. This example contains the design of a typical interior beam at the critical sections in positive flexure, shear, and deflection. The superstructure consists of six beams spaced at 9'-0" centers. A typical transverse superstructure section is provided in Figure 5.7.2.1. A framing plan is provided in Figure 5.7.2.2. The roadway section is composed of two 12' traffic lanes and two 12' shoulders. A Type F railing is provided on each side of the bridge and a 9" composite concrete deck is used. End diaphragms (B814) are used at each end of the bridge and interior diaphragms (B403) are used at the interior third points and at the pier.



SECTION A-A

**Figure 5.7.2.1**

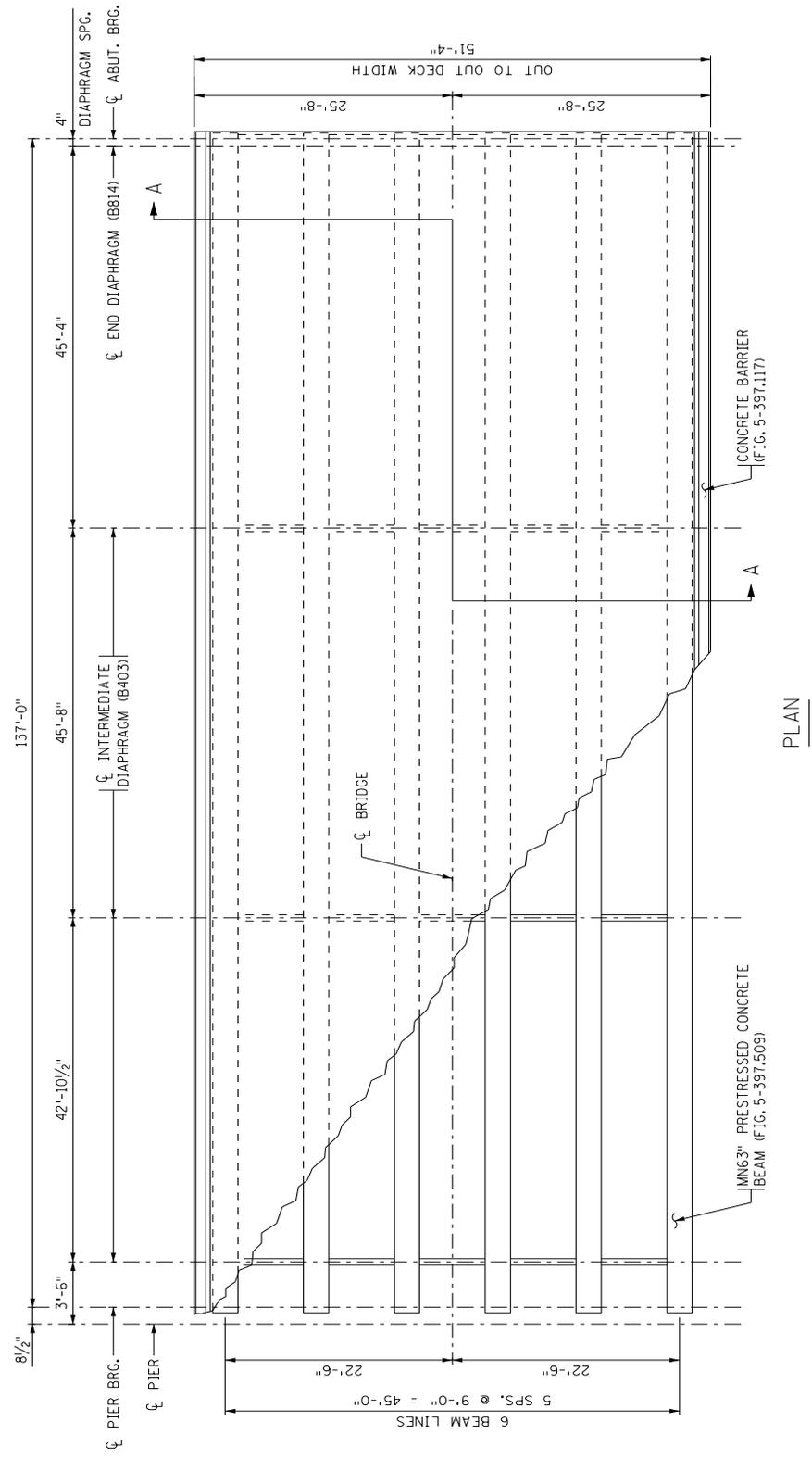


Figure 5.7.2.2

**A. Materials**

The modulus of elasticity for high strength concrete suggested by ACI Committee 363 is used for the beam concrete. The composite deck is assumed to have a unit weight of 0.150 kcf for dead load computations and 0.145 kcf for elastic modulus computations. The beam concrete is assumed to have a unit weight of 0.155 kcf for dead load computations.

The material and geometric parameters used in the example are shown in Table 5.7.2.1:

**Table 5.7.2.1  
Material Properties**

Material Parameter		Prestressed Beam	Deck
Concrete	$f_{ci}$ at transfer	7.5 ksi	---
	$f_c$ at 28 days	9.0 ksi	4 ksi
	$E_{ci}$ at transfer	$(1265 \cdot \sqrt{f_{ci}}) + 1000$ = 4464 ksi	---
	$E_c$ at 28 days	$(1265 \cdot \sqrt{f_c}) + 1000$ = 4795 ksi	$33,000 \cdot (0.145)^{1.5} \cdot \sqrt{f_c}$ = 3644 ksi
Steel	$f_y$ for rebar	60 ksi	60 ksi
	$f_{pu}$ for strand	270 ksi	---
	$E_s$ for rebar	29,000 ksi	29,000 ksi
	$E_p$ for strand	28,500 ksi	---
	Strand type	0.6 inch diameter 270 ksi, low relaxation	---

**B. Determine Cross-Section Properties for a Typical Interior Beam**

The beams are designed to act compositely with the deck on simple spans. The deck consists of a 7 inch thick concrete slab with a 2 inch wearing course. For simplicity and in order to be conservative, the beams are designed assuming the full 9 inches of thickness is placed in a single pour. A 1/2 inch of wear is assumed. A thickness of 8 1/2 inches is used for composite section properties. The haunch or stool is assumed to have an average thickness of 2 1/2 inches for dead load computations and 1 1/2 inches for section property computations.

**[4.6.2.6.1]**

The effective flange width,  $b_e$ , is equal to the average beam spacing:  
 $b_e = 108.0$  in

The modular ratio of the deck concrete to the beam concrete is:

$$n = \frac{E_{cdeck}}{E_{cbeam}} = \frac{3644}{4795} = 0.76$$

This results in a transformed effective flange width of:

$$b_{\text{etrans}} = 0.76 \cdot (108.0) = 82.1 \text{ in}$$

Properties for an interior beam are given in Table 5.7.2.2.

**Table 5.7.2.2**  
**Cross-Section Properties**

Parameter	Non-composite Section	Composite Section
Height of section, h	63 in	73.0 in
Deck thickness	---	8.5 in
Average stool thickness	---	1.5 in (section properties) 2.5 in (dead load)
Effective flange width, $b_e$	---	108.0 in (deck concrete) 82.1 in (beam concrete)
Area, A	807 in <sup>2</sup>	1543 in <sup>2</sup>
Moment of inertia, I	422,570 in <sup>4</sup>	1,034,168 in <sup>4</sup>
Centroidal axis height, $y$	28.80 in	47.74 in
Bottom section modulus, $S_b$	14,673 in <sup>3</sup>	21,664 in <sup>3</sup>
Top section modulus, $S_t$	12,356 in <sup>3</sup>	53,862 in <sup>3</sup> (beam concrete)
Top of prestressed beam, $S_{\text{tbm}}$	12,356 in <sup>3</sup>	67,753 in <sup>3</sup>

### **C. Shear Forces and Bending Moments**

Three load combinations will be considered; Strength I, Service I, and Service III. As a result of the simple span configuration, only maximum  $\gamma_p$  values need to be considered.

Load effects related to settlement, thermal effects, water load, or stream pressure will not be considered.

Assume that traffic can be positioned anywhere between the barriers.

$$\text{Number of design lanes} = \frac{\text{distance between barriers}}{\text{design lane width}} = \frac{48}{12} = 4$$

[3.6.2] Dynamic load allowance IM = 33%

[4.6.2.2] **1. Determine Live Load Distribution Factors**

Designers should note that the approximate distribution factor equations include the multiple presence factors.

[4.6.2.2.2] **Distribution Factor for Moment – Interior Beams**

LRFD Table 4.6.2.2.1-1 lists the common deck superstructure types for which approximate live load distribution equations have been assembled. The cross section for this design example is Type (k). To ensure that the approximate distribution equations can be used, several parameters need to be checked.

- 1) 3.5 ft ≤ beamspacing = 9.0 ft ≤ 16.0 ft OK
- 2) 4.5 in ≤ slabthickness = 8.5 in ≤ 12.0 in OK
- 3) 20 ft ≤ span length = 130 ft ≤ 240 ft OK
- 4) 4 ≤ numberofbeams = 6 OK

The distribution factor equations use a  $K_g$  factor that is defined in LRFD Article 4.6.2.2.1.

$$\eta = \frac{E_{c \text{ beam}}}{E_{c \text{ deck}}} = \frac{4795}{3644} = 1.316$$

$$e_g = (\text{deck centroid}) - (\text{beam centroid}) = 68.75 - 28.80 = 39.95 \text{ in}$$

$$K_g = \eta \cdot [I + A \cdot (e_g)^2] = 1.316 \cdot [422,570 + 807 \cdot (39.95)^2] = 2.25 \times 10^6$$

One design lane loaded:

$$gM = 0.06 + \left(\frac{S}{14}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K}{12 \cdot L \cdot t_s^3}\right)^{0.1}$$

$$gM = 0.06 + \left(\frac{9.0}{14}\right)^{0.4} \cdot \left(\frac{9.0}{137}\right)^{0.3} \cdot \left(\frac{2.25 \times 10^6}{12 \cdot 137 \cdot 8.5^3}\right)^{0.1}$$

$$gM = 0.461 \text{ lanes/beam}$$

Two or more design lanes loaded:

$$gM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{12 \cdot L \cdot t_s^3}\right)^{0.1}$$

$$gM = 0.075 + \left(\frac{9.0}{9.5}\right)^{0.6} \cdot \left(\frac{9.0}{137}\right)^{0.2} \cdot \left(\frac{2.25 \times 10^6}{12 \cdot 137 \cdot 8.5^3}\right)^{0.1}$$

$$gM = 0.683 \text{ lanes/beam}$$

**[4.6.2.2.2d]**

**Distribution Factor for Moment - Exterior Beams**

LRFD Table 4.6.2.2.2d-1 contains the approximate distribution factor equations for exterior beams. Type (k) cross-sections have a deck dimension check to ensure that the approximate equations are valid.

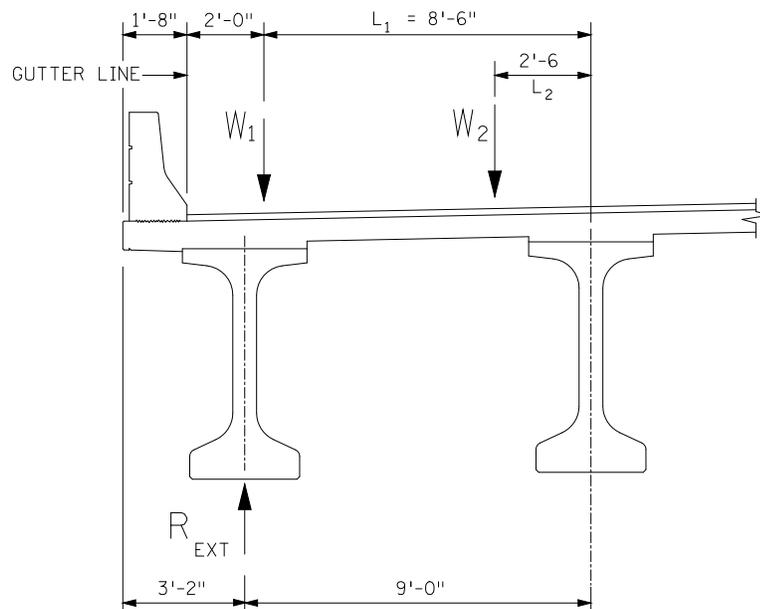
The distance from the inside face of barrier to the centerline of the fascia beam is defined as  $d_e$ . For the example this distance is:

$$d_e = 24 - (2.5 \cdot 9.0) = 1.50 \text{ ft}$$

The check to use the approximate equations is:

$$-1.0 \text{ ft} \leq d_e = 1.50 \text{ ft} \leq 5.5 \text{ ft} \quad \text{OK}$$

One design lane loaded:



**Figure 5.7.2.3**

Use the lever rule to determine the live load distribution factor for one lane.

The exterior beam live load distribution factor is found by determining the exterior beam reaction:

$$W_1 = W_2 = 0.5 \text{ lanes}$$

$$gM = 1.2 \cdot \left( \frac{W_1 \cdot L_1 + W_2 \cdot L_2}{S} \right) = 1.2 \cdot \left( \frac{0.5 \cdot 8.5 + 0.5 \cdot 2.5}{9.0} \right)$$

$$gM = 0.733 \text{ lanes/beam}$$

Two or more design lanes loaded:

The distribution factor is equal to the factor "e" multiplied by the interior girder distribution factor for two or more lanes

$$e = 0.77 + \left( \frac{d_e}{9.1} \right) = 0.77 + \left( \frac{1.5}{9.1} \right) = 0.935$$

$$gM = e \cdot g_{int} = 0.935 \cdot 0.683 = 0.639 \text{ lanes/beam}$$

#### [4.6.2.2.2e]

#### Skew Factor

No correction is necessary for a skew angle of zero.

#### [4.6.2.2.3]

#### [4.6.2.2.3a]

#### Distribution Factor for Shear – Interior Beams

LRFD Table 4.6.2.2.3a-1 can be used.

One design lane loaded:

$$gV = 0.36 + \left( \frac{S}{25.0} \right) = 0.36 + \left( \frac{9.0}{25.0} \right) = 0.720 \text{ lanes/beam}$$

Two or more design lanes loaded:

$$gV = 0.2 + \left( \frac{S}{12} \right) - \left( \frac{S}{35} \right)^2 = 0.2 + \left( \frac{9.0}{12} \right) - \left( \frac{9.0}{35} \right)^2 = 0.884 \text{ lanes/beam}$$

#### [4.6.2.2.3b]

#### Distribution Factor for Shear – Exterior Beams

One Design Lane Loaded:

Use the lever rule, which results in the same factor that was computed for flexure and is equal to 0.733 lanes/beam

Two or more design lanes loaded:

$$e = 0.6 + \left( \frac{d_e}{10} \right) = 0.6 + \left( \frac{1.5}{10} \right) = 0.750$$

The exterior beam shear distribution factor for two or more design lanes is determined by modifying the interior distribution factor:

$$gV = e \cdot g_{int} = 0.750 \cdot 0.884 = 0.663 \text{ lanes/beam}$$

**[4.6.2.2.3c]**

**Skew Factor**

No correction is necessary for a skew angle of zero.

**[2.5.2.6.2]**

**[Table 3.6.1.1.2-1]**

**Distribution Factor for Deflection**

The distribution factor for checking live load deflections assumes that the entire cross section participates in resisting the live load. The minimum Multiple Presence Factor (MPF) used by MnDOT when checking live load deflection is 0.85. The deflection distribution factor is:

$$gD = \frac{(\# \text{ of lanes}) \cdot (\text{MPF})}{(\# \text{ of beam lines})} = \frac{4 \cdot 0.85}{6} = 0.567 \text{ lanes/beam}$$

Table 5.7.2.3 contains a summary of the live load distribution factors.

**Table 5.7.2.3**

**Distribution Factor Summary (lanes per beam)**

Loading		Flexure	Shear
Interior Beam	One Design Lane	0.461	0.720
	Two or More Design Lanes	0.683	0.884
	Deflection	0.567	-
Exterior Beam	One Design Lane	0.733	0.733
	Two or More Design Lanes	0.639	0.663
	Deflection	0.567	-

**[1.3.3 – 1.3.5]**

**2. Load Modifiers**

The following load modifiers will be used for this example:

		Strength	Service	Fatigue
Ductility	$\eta_D$	1.0	1.0	1.0
Redundancy	$\eta_R$	1.0	1.0	1.0
Importance	$\eta_I$	1.0	n/a	n/a
	$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.0	1.0	1.0

**3. Dead and Live Load Summary**

$$\text{Beam Selfweight} = (807/144) \cdot (0.155 \text{ kip/ft}^2) = 0.869 \text{ kip/ft}$$

$$\text{Stool Weight} = (2.83 \text{ ft}) \cdot (0.208 \text{ ft}) \cdot (0.150 \text{ kip/ft}^2) = 0.088 \text{ kip/ft}$$

$$\text{Deck Weight} = (9.0 \text{ ft}) \cdot (0.75 \text{ ft}) \cdot (0.150 \text{ kip/ft}^2) = 1.013 \text{ kip/ft}$$

$$\text{Future Wearing Surface} = (0.020 \text{ kip/ft}^2) \cdot (48 \text{ ft}) \cdot (1/6) = 0.160 \text{ kip/ft}$$

$$\text{Barrier Weight} = 2 \cdot (0.439 \text{ kip/ft}) \cdot (1/6) = 0.146 \text{ kip/ft}$$

$$\text{Diaphragm Weight} \cong (9.0) \cdot (0.0427)$$

$$+ 2 \cdot (1.75) \cdot (1.0) \cdot \left(\frac{0.375}{12}\right) \cdot (0.490) = 0.44 \text{ kip}$$

The bending moments and shears for the dead and live loads were obtained with a line girder model of the bridge. They are summarized in Tables 5.7.2.4 and 5.7.2.5.

**Table 5.7.2.4  
Shear Force Summary (kips/beam)**

Load Type/Combination		Brg CL (0.0')	Brg Face (0.63')	Trans Point (2.38')	Critical Shear Point (5.03')	0.1 Span Point (13.7')	0.2 Span Point (27.4')	0.3 Span Point (41.1')	0.4 Span Point (54.8')	0.5 Span Point (68.5')
Dead Loads	Selfweight	60	59	57	55	48	36	24	12	0
	Stool	6	6	6	6	5	4	2	1	0
	Deck	69	69	67	64	56	42	28	14	0
	FWS	11	11	11	10	9	7	4	2	0
	Barrier	10	10	10	9	8	6	4	2	0
	Diaphragms	0	0	0	0	0	0	0	0	0
	Total	156	155	151	144	126	95	62	31	0
Live Loads	Uniform Lane	39	39	37	36	31	25	19	14	10
	Truck with DLA	78	78	77	75	70	62	53	45	36
	Total	117	117	114	111	101	87	72	59	46
Strength I Load Comb (1.25 · DL + 1.75 · LL)		401	399	388	376	335	269	205	142	81
Service I Load Comb (1.00 · DL + 1.00 · LL)		273	272	265	255	227	182	134	90	46
Service III Load Comb (1.00 · DL + 0.80 · LL)		250	249	242	233	207	165	120	78	37

**Table 5.7.2.5  
Bending Moment Summary (kip-ft/beam)**

Load Type/Combination		Brg CL (0.0')	Brg Face (0.63')	Trans Point (2.38')	Critical Shear Point (5.03')	0.1 Span Point (13.7')	0.2 Span Point (27.4')	0.3 Span Point (41.1')	0.4 Span Point* (54.8')	0.5 Span Point (68.5')	
Dead Loads	DC1	Selfweight	0	37	139**	288	734	1305	1713	1957	2039
		Stool	0	4	14	29	74	132	173	198	206
		Deck	0	43	162	336	856	1521	1996	2282	2377
		Diaphragms	0	0	1	2	6	12	18	20	20
		Total DC1	0	84	316	655	1670	2970	3900	4457	4642
	DC2	Barrier	0	6	23	48	123	219	288	329	343
		FWS	0	7	26	53	135	240	315	360	375
		Total DC2	0	13	49	101	258	459	603	689	718
	Total (DC1+DC2)		0	97	365	756	1928	3429	4503	5146	5360
	Live Loads	Uniform Lane	0	19	70	145	369	656	861	985	1026
Truck with DLA		0	38	142	294	746	1312	1699	1927	1986	
Total		0	57	212	439	1115	1968	2560	2912	3012	
Strength I - Load Comb (1.25 · DL + 1.75 · LL)		0	221	827	1713	4361	7730	10109	11529	11971	
Service I - Load Comb (1.00 · DL + 1.00 · LL)		0	154	577	1195	3042	5397	7063	8058	8372	
Service III - Load Comb (1.00 · DL + 0.80 · LL)		0	143	535	1107	2820	5003	6551	7476	7770	

\* Drape point for strands.

\*\* Beam selfweight at strand release = 176 k-ft

Beam selfweight at erection on bearings = 139 k-ft

**D. Design  
Prestressing**

Typically the tension at the bottom of the beam at midspan dictates the required level of prestressing.

**1. Estimate Required Prestress**

Use the Service III load combination

Bottom of beam stress:

$$\begin{aligned}
 &= \left( \frac{M_{DC1}}{S_{gb}} \right) + \left( \frac{M_{DC2}}{S_{cb}} \right) + \left( \frac{M_{LL} \cdot 0.8}{S_{cb}} \right) \\
 &= \left( \frac{4642 \cdot 12}{14,673} \right) + \left( \frac{718 \cdot 12}{21,664} \right) + \left( \frac{3012 \cdot 12 \cdot 0.8}{21,664} \right) = 5.53 \text{ ksi}
 \end{aligned}$$

As a starting point, the total prestress losses will be assumed to be 25%. This results in an effective prestress of

$$f_{pe} = 0.75 \cdot f_{pu} \cdot (1 - 0.25) = 0.75 \cdot 270 \cdot 0.75 = 151.9 \text{ ksi}$$

Strands are typically placed on a 2" grid. The bottom flange of an "MN63" beam can hold a maximum of 54 strands. The centroid of a 54 strand pattern would be

$$y_{str} = \left[ \frac{\sum (\# \text{ of strands}) \cdot (\gamma \text{ of strands})}{(\text{total} \# \text{ of strands})} \right]$$

$$= \left[ \frac{12 \cdot (2 + 4 + 6) + (6 \cdot 8) + 2 \cdot (3 + 5 + 7 + 9 + 11 + 13)}{54} \right] = 5.33 \text{ in}$$

Using the centroid of this group as an estimate of the strand pattern eccentricity results in

$$e_{54} = y_g - 5.33 = 28.80 - 5.33 = 23.47 \text{ in}$$

The area of a 0.6" diameter 7-wire strand is 0.217 in<sup>2</sup>

The axial compression produced by the prestressing strands is

$$P = A_s \cdot f_{pe} = (\# \text{ of strands}) \cdot (0.217) \cdot (151.9)$$

The internal moment produced by the prestressing strands is

$$M_{p/s} = A_s \cdot f_{pe} \cdot e_{54} = (\# \text{ of strands}) \cdot 0.217 \cdot 151.9 \cdot 23.47$$

The allowable tension after losses =  $0.19 \cdot \sqrt{f'_c} = 0.19 \cdot \sqrt{9} = 0.57 \text{ ksi}$

This moment and the axial compression from the prestress must reduce the bottom flange tension from 5.53 ksi tension to a tension of 0.57 ksi or

$$\text{Required } f_{pe} = 5.53 - 0.57 = 4.96 \text{ ksi}$$

$$\text{Using the fact that } f_{pe} = \frac{P}{A} + \frac{M}{S}$$

One can estimate the required number of strands:

$$\frac{\left[ \frac{4.96}{\left( \frac{1}{A_g} + \frac{23.47}{S_{gb}} \right)} \right]}{(0.217 \cdot 1519)} = \frac{\left[ \frac{4.96}{\left( \frac{1}{807} + \frac{23.47}{14,673} \right)} \right]}{(0.217 \cdot 1519)} = 53.0 \text{ strands}$$

Try a strand pattern with 54 strands.

After reviewing Bridge Details Part II Figure 5-397.509, a 54 strand draped strand pattern was selected. Also, the drape points were chosen to be at  $0.40L = 54.8$  ft from the centerline of bearing locations. The trial strand pattern is shown in Figure 5.7.2.4.

The properties of this strand pattern at midspan are:

$$y_{strand} = \left[ \frac{12 \cdot (2 + 4 + 6) + (4 \cdot 8) + 2 \cdot (3 + 5 + 7 + 9 + 11 + 13 + 15)}{54} \right] = 5.59 \text{ in}$$

$$e_{strand} = y_b - y_{strand} = 28.80 - 5.59 = 23.21 \text{ in}$$

Section Modulus at the strand pattern centroid is

$$S_{gps} = \frac{I_g}{e_{strand}} = \frac{422,570}{23.21} = 18,206 \text{ in}^3$$

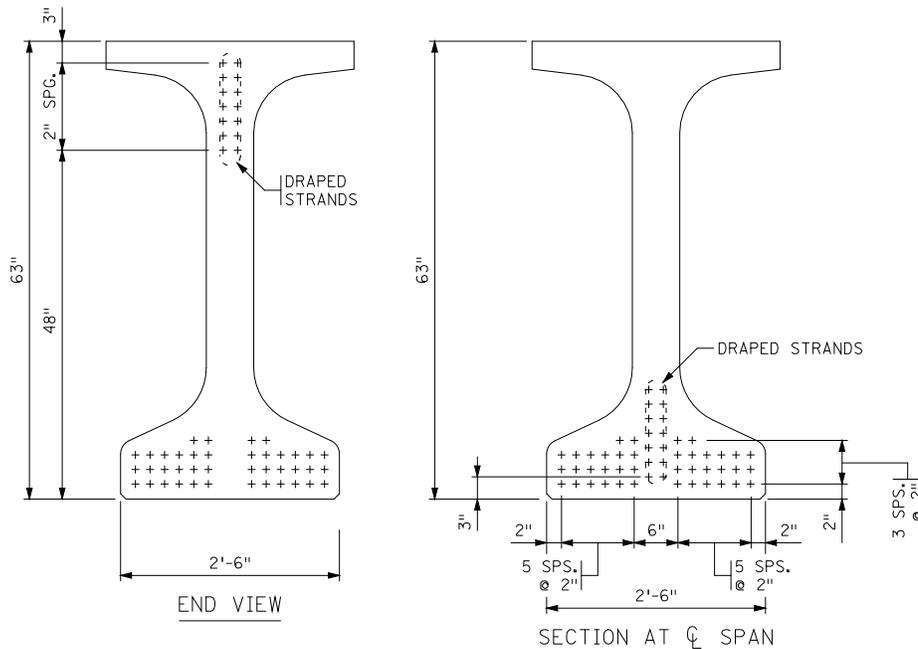


Figure 5.7.2.4

**[5.9.5]****2. Prestress Losses**

Prestress losses are computed using the approximate method.

**[5.9.5.2.3]****Elastic Shortening Loss**

Use the alternative equation presented in the LRFD C5.9.5.2.3a.

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt} \cdot (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \frac{A_g I_g E_{ci}}{E_p}}$$

$$A_{ps} = (\# \text{ of strands}) \cdot (\text{strand area}) = 54 \cdot 0.217 = 11.72 \text{ in}^2$$

$$f_{pbt} = f_{pj} = 202.50 \text{ ksi}$$

$$e_m = e_{\text{strand}} = 23.21 \text{ in}$$

$$\frac{A_g I_g E_{ci}}{E_p} = \frac{807(422,570)(4464)}{28,500} = 53,413,560 \text{ in}^6$$

$$A_{ps} (I_g + e_m^2 A_g) = 11.72 [422,570 + (23.2)^2 (807)] = 10,047,605 \text{ in}^6$$

$$\Delta f_{pES} = \frac{202.5 \cdot (10,047,605) - 23.21 (2039) (12) (807)}{10,047,605 + 53,413,560} = 24.8 \text{ ksi}$$

**[5.9.5.3]****Long Term Losses**

Use the approximate equation in the LRFD 5.9.5.3

$$\Delta f_{pLT} = 10.0 \cdot \frac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR}$$

For an average humidity in Minnesota of 73%

$$\gamma_h = 1.7 - 0.01 \cdot H = 1.7 - 0.01 \cdot 73 = 0.97$$

$$\gamma_{st} = \frac{5}{1 + f'_{ci}} = \frac{5}{1 + 7.5} = 0.59$$

$$\Delta f_{pLT} = 10.0 \cdot \frac{202.5 \cdot (11.72)}{807} \cdot 0.97(0.59) + 12.0(0.97)(0.59) + 2.4 = 26.1 \text{ ksi}$$

**[5.9.5.1]****Total Losses**

$$\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pLT} = 24.8 + 26.1 = 50.9 \text{ ksi}$$

$$f_{pe} = f_{pj} - \Delta f_{pt} = 202.5 - 50.9 = 151.6 \text{ ksi}$$

$$\text{prestress loss percentage} = \frac{\Delta f_{pt}}{f_{pj}} \cdot 100 = \frac{50.9}{202.50} \cdot 100 = 25.1\%$$

initial prestress force

$$P_i = A_{ps} \cdot (f_{pj} - \Delta f_{pES}) = 11.72 \cdot (202.5 - 24.8) = 2083 \text{ kips}$$

prestress force after all losses

$$P_e = A_{ps} \cdot f_{pe} = 11.72 \cdot 151.6 = 1777 \text{ kips}$$

### [5.9.4.1]

### 3. Stresses at Transfer (compression +, tension -)

#### Stress Limits for P/S Concrete at Release

Compression in the concrete is limited to:

$$0.60 \cdot f'_{ci} = 0.60 \cdot 7.5 = 4.50 \text{ ksi}$$

Tension in the concrete is limited to:

$$\begin{aligned} \text{The minimum of } -0.0948 \cdot \sqrt{f'_{ci}} &= -0.0948 \cdot \sqrt{7.5} = -0.26 \text{ ksi} \\ &\text{or } -0.20 \text{ ksi} \end{aligned}$$

Tension limit = -0.20 ksi

#### Check Release Stresses at Drape Point (0.40 Point of Span)

The selfweight moment is calculated using the design length to simplify calculations. This is conservative for calculation of release stresses.

$$P_i \cdot e_{\text{strand}} = 2083 \cdot 23.21 = 48,346 \text{ kip-in}$$

$$\begin{aligned} \text{Top stress due to P/S} &= \left( \frac{P_i}{A_g} \right) - \left( \frac{P_i \cdot e_{\text{strand}}}{S_{gt}} \right) = \left( \frac{2083}{807} \right) - \left( \frac{48,346}{12,356} \right) \\ &= -1.33 \text{ ksi} \end{aligned}$$

$$\begin{aligned} \text{Bottom Stress due to P/S} &= \left( \frac{P_i}{A_g} + \frac{P_i \cdot e_{\text{strand}}}{S_{gb}} \right) = \left( \frac{2083}{807} \right) + \left( \frac{48,346}{14,673} \right) \\ &= 5.88 \text{ ksi} \end{aligned}$$

$$\text{Selfweight moment at drape point} = M_{sw0.40} = 1957 \text{ kip-ft}$$

$$\text{Top stress due to selfweight} = \left( \frac{M_{sw0.40}}{S_{gt}} \right) = \left( \frac{1957 \cdot 12}{12,356} \right) = 1.90 \text{ ksi}$$

$$\text{Bottom stress due to selfweight} = \left( \frac{M_{sw0.40}}{S_{gb}} \right) = \left( \frac{1957 \cdot 12}{14,673} \right) = -1.60 \text{ ksi}$$

$$\text{Top stress at drape point} = -1.33 + 1.90 = 0.57 \text{ ksi} < -0.20 \text{ ksi}$$

OK

$$\text{Bottom stress at drape point} = 5.88 - 1.60 = 4.28 \text{ ksi} < 4.50 \text{ ksi}$$

OK

**Check Release Stresses at End of Beam**

The strands need to be draped to raise the eccentricity of the prestress force and limit the potential for cracking the top of the beams. Stresses are checked at the transfer point (60 bar diameters from the end of the beam), using the total length of the beam for self weight moment calculations.

Centroid of strand pattern at the end of the beams:

$$y_{\text{strand}} = \left[ \frac{12 \cdot (2 + 4 + 6) + (4 \cdot 8) + 2 \cdot (60 + 58 + 56 + 54 + 52 + 50 + 48)}{54} \right]$$

$$= 17.26 \text{ in}$$

Centroid of strand at the transfer point:

$$y_{\text{strand}} = 17.26 - \frac{60 \cdot 0.6}{137 \cdot 0.4 \cdot 12 + \frac{15}{2}} \cdot (17.26 - 5.59) = 16.63 \text{ in}$$

The eccentricity of the strand pattern is:

$$e_{\text{strand}} = y_b - y_{\text{strand}} = 28.80 - 16.63 = 12.17 \text{ in}$$

The internal prestress moment is:

$$P_i \cdot e_{\text{strand}} = 2083 \cdot 12.17 = 25,350 \text{ kip-in}$$

$$\text{Top stress due to P/S} = \left( \frac{P_i}{A_g} \right) - \left( \frac{P_i \cdot e_{\text{strand}}}{S_{gt}} \right) = \left( \frac{2083}{807} \right) - \left( \frac{25,350}{12,356} \right)$$

$$= 0.53 \text{ ksi}$$

$$\text{Bottom stress due to P/S} = \left( \frac{P_i}{A_g} \right) + \left( \frac{P_i \cdot e_{\text{strand}}}{S_{gb}} \right) = \left( \frac{2083}{807} \right) + \left( \frac{25,350}{14,673} \right)$$

$$= 4.31 \text{ ksi}$$

$$\text{Top stress due to selfweight} = \left( \frac{M_{\text{swtr}}}{S_{gt}} \right) = \left( \frac{176 \cdot 12}{12,356} \right) = 0.17 \text{ ksi}$$

$$\text{Bottom stress due to selfweight} = - \left( \frac{M_{\text{swtr}}}{S_{gb}} \right) = - \left( \frac{176 \cdot 12}{14,673} \right) = -0.14 \text{ ksi}$$

Top stress at transfer point = 0.53 + 0.17 = 0.70 ksi < -0.20 ksi

OK

Bottom stress at transfer point =  $4.31 - 0.14 = 4.17$  ksi < 4.50 ksi OK

By back calculating the highest compressive stress at release, located at the drape point, it is found that the concrete compressive strength at release can be reduced:

$$f'_{ci} = \frac{4.28}{0.60} = 7.13 \text{ ksi} \quad \text{try } f'_{ci} = 7.2 \text{ ksi}$$

When modifying the initial concrete strength, prestress losses must be recalculated. The new losses are summarized below.

$$\Delta f_{pES} = 25.0 \text{ ksi} \quad \text{p/s loss \% at release} = 12.3\%$$

$$\Delta f_{pLT} = 26.9 \text{ ksi}$$

$$\Delta f_{pt} = 51.9 \text{ ksi} \quad \text{p/s loss \% after all losses} = 25.6\%$$

initial prestress force

$$P_i = A_{ps} \cdot (f_{pj} - \Delta f_{pES}) = 11.72 \cdot (202.5 - 25.0) = 2080 \text{ kips}$$

prestress force after all losses

$$f_{pe} = f_{pj} - \Delta f_{pt} = 202.5 - 51.9 = 150.6 \text{ ksi}$$

$$P_e = A_{ps} \cdot f_{pe} = 11.72 \cdot 150.6 = 1765 \text{ kips}$$

The new bottom stress at the drape point =  $4.27 < 0.60 \cdot f'_{ci} = 4.32$  ksi

OK

#### [5.9.4.2]

#### 4. Stresses at Service Loads (compression +, tension -) Stress Limits for P/S Concrete after All Losses

Compression in the concrete is limited to (Service I Load Combination):

$$0.45 \cdot f'_c = 0.45 \cdot 9.0 = 4.05 \text{ ksi}$$

(for prestress and permanent loads)

Check the bottom stress at end of beam and the top stress at midspan against this limit.

$$0.40 \cdot f'_c = 0.40 \cdot 9.0 = 3.60 \text{ ksi}$$

(for live load and  $\frac{1}{2}$  of prestress and permanent loads)

Check the top stress at midspan against this limit.

$$0.60 \cdot Q_w \cdot f'_c = 0.60 \cdot 1.0 \cdot 9.0 = 5.40 \text{ ksi}$$

(for live load, prestress, permanent loads, and transient loads)

Check the top stress at midspan against this limit.

Tension in the concrete is limited to (Service III Load Combination):

$$-0.19 \cdot \sqrt{f'_c} = -0.19 \cdot \sqrt{9.0} = -0.570 \text{ ksi}$$

Check the bottom stress at midspan against this limit.

**Check Stresses at Midspan After Losses:**

Bottom stress

$$\begin{aligned} &= -\left(\frac{M_{DC1}}{S_{gb}}\right) - \left(\frac{M_{DC2}}{S_{cb}}\right) - \left(\frac{M_{LL} \cdot 0.8}{S_{cb}}\right) + \left(\frac{P_e}{A_g}\right) + \left(\frac{P_e \cdot e_{strand}}{S_{gb}}\right) \\ &= -\left(\frac{4642 \cdot 12}{14,673}\right) - \left(\frac{718 \cdot 12}{21,664}\right) - \left(\frac{3012 \cdot 12 \cdot 0.8}{21,664}\right) + \left(\frac{1765}{807}\right) + \left(\frac{1765 \cdot 23.21}{14,673}\right) \\ &= -0.550 \text{ ksi} < -0.570 \text{ ksi} \end{aligned} \quad \text{OK}$$

Top stress due to all loads

$$\begin{aligned} &= \left(\frac{P_e}{A_g}\right) - \left(\frac{P_e \cdot e_{strand}}{S_{gt}}\right) + \left(\frac{M_{DC1}}{S_{gt}}\right) + \left(\frac{M_{DC2} + M_{LL}}{S_{gtc}}\right) \\ &= \left(\frac{1765}{807}\right) - \left(\frac{1765 \cdot 23.21}{12,356}\right) + \left(\frac{4642 \cdot 12}{12,356}\right) + \left[\frac{(718 + 3012) \cdot 12}{67,753}\right] \\ &= 4.04 \text{ ksi} < 5.40 \text{ ksi} \end{aligned} \quad \text{OK}$$

Top stress due to permanent loads

$$\begin{aligned} &= \left(\frac{P_e}{A_g}\right) - \left(\frac{P_e \cdot e_{strand}}{S_{gt}}\right) + \left(\frac{M_{DC1}}{S_{gt}}\right) + \left(\frac{M_{DC2}}{S_{gtc}}\right) \\ &= \left(\frac{1765}{807}\right) - \left(\frac{1765 \cdot 23.21}{12,356}\right) + \left(\frac{4642 \cdot 12}{12,356}\right) + \left(\frac{718 \cdot 12}{67,753}\right) \\ &= 3.51 \text{ ksi} < 4.05 \text{ ksi} \end{aligned} \quad \text{OK}$$

Top stress due to live load plus 1/2 of prestress and permanent loads

$$\begin{aligned} &= \frac{1}{2} \left( \left(\frac{P_e}{A_g}\right) - \left(\frac{P_e \cdot e_{strand}}{S_{gt}}\right) + \left(\frac{M_{DC1}}{S_{gt}}\right) + \left(\frac{M_{DC2}}{S_{gtc}}\right) \right) + \left(\frac{M_{LL}}{S_{gtc}}\right) \\ &= \frac{1}{2} \left( \left(\frac{1765}{807}\right) - \left(\frac{1765 \cdot 23.21}{12,356}\right) + \left(\frac{4642 \cdot 12}{12,356}\right) + \left(\frac{718}{67,753}\right) \right) + \left(\frac{3012 \cdot 12}{67,753}\right) \\ &= 2.23 \text{ ksi} < 3.60 \text{ ksi} \end{aligned} \quad \text{OK}$$

**Check the Compression Stresses at End of Beam After Losses**

Bottom flange stress due to prestress and permanent loads. For simplicity, all loads are ignored for this calculation, which is conservative.

$$\frac{P_e}{A_g} + \frac{P_e \cdot e_{\text{strand}}}{S_{gb}} = \frac{1765}{807} + \frac{1765 \cdot 12.17}{14,673} = 3.65 \text{ ksi} < 4.05 \text{ ksi} \quad \underline{\text{OK}}$$

The final concrete stress may also be reduced by back calculating the bottom tensile stress under the Service III load condition. In this instance:

$$\text{Min } f'_c = \left( \frac{0.550}{0.19} \right)^2 = 8.38 \text{ ksi} \quad \text{Try } f'_c = 8.50 \text{ ksi}$$

This change will not effect the computed losses, but the actual and allowable stresses must be recomputed and compared.

At midspan:

$$\text{Bottom stress due to all loads} = -0.545 \text{ ksi} < -0.554 \text{ ksi} \quad \underline{\text{OK}}$$

(Service III)

$$\text{Top stress due to all loads} = 4.02 \text{ ksi} < 5.10 \text{ ksi} \quad \underline{\text{OK}}$$

$$\text{Top stress due to permanent loads} = 3.51 \text{ ksi} < 3.83 \text{ ksi} \quad \underline{\text{OK}}$$

$$\begin{aligned} \text{Top stress due to live load plus } \frac{1}{2} \text{ (prestress + permanent loads)} \\ = 2.27 \text{ ksi} < 3.40 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

Therefore, use  $f'_{ci} = 7.2 \text{ ksi}$  and  $f'_c = 8.5 \text{ ksi}$

**[5.5.4]****5. Flexure – Strength Limit State**

Resistance factors at the strength limit state are:

$$\phi = 1.00 \text{ for flexure and tension (assumed)}$$

$$\phi = 0.90 \text{ for shear and torsion}$$

$$\phi = 1.00 \text{ for tension in steel in anchorage zones}$$

Strength I design moment is 11,971 kip-ft at midspan.

From previous calculations, distance to strand centroid from bottom of the beam is:

$$Y_{\text{strand}} = 5.59 \text{ in}$$

$$k = 2 \cdot \left( 1.04 - \frac{f_{py}}{f_{pu}} \right) = 2 \cdot \left( 1.04 - \frac{243}{270} \right) = 0.280$$

$$d_p = (\text{beamheight}) + \text{stool} + \text{deck} - Y_{\text{strand}}$$

**[5.7.3.1.1]**

$$= 63 + 1.5 + 8.5 - 5.59 = 67.41 \text{ in}$$

[5.7.3.1.1-4]

$$c = \left[ \frac{(A_{ps} \cdot f_{pu})}{\left( 0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p} \right)} \right]$$

$$= \left( \frac{11.72 \cdot 270}{0.85 \cdot 4.0 \cdot 0.85 \cdot 108 + 0.28 \cdot 11.72 \cdot \frac{270}{67.41}} \right) = 9.73 \text{ in}$$

$$f_{ps} = f_{pu} \cdot \left( 1 - k \cdot \frac{c}{d_p} \right) = 270 \cdot \left( 1 - 0.28 \cdot \frac{9.73}{67.41} \right) = 259.1 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.85 \cdot 9.73 = 8.27 \text{ in}$$

Compression block depth is less than 8.5", the thickness of the slab, so T-section behavior is not considered.

Internal lever arm between compression and tension flexural force components:

$$d_p - \frac{a}{2} = 67.41 - \frac{8.27}{2} = 63.28 \text{ in}$$

$$M_n = A_{ps} \cdot f_{ps} \cdot 63.28 = 11.72 \cdot 259.1 \cdot 63.28 = 192,159 \text{ kip-in}$$

$$= 16,013 \text{ kip-ft}$$

$$\phi M_n = 1.0 \cdot 16,013 = 16,013 \text{ kip-ft} > M_u = 11,971 \text{ kip-ft} \quad \text{OK}$$

[5.5.4.2.1]

Validate the assumption of 1.0 for the resistance factor:

$$\phi = 0.583 + 0.25 \cdot \left( \frac{d_t}{c} - 1 \right) = 0.583 + 0.25 \cdot \left( \frac{67.41}{9.73} - 1 \right) = 2.07 > 1$$

Therefore  $\phi = 1.0$ , which matches the assumption

[5.7.3.3.2]

### 6. Minimum Reinforcement

$$f_r = 0.37 \cdot \sqrt{f'_c} = 0.37 \cdot \sqrt{8.5} = 1.08 \text{ ksi}$$

$$f_{peb} = \frac{P_e}{A_g} + \frac{P_e \cdot e_{strand}}{S_{gb}}$$

$$= \frac{1765}{807} + \frac{1765 \cdot 23.21}{14,673} = 4.98 \text{ ksi}$$

$$M_{cr} = (f_r + f_{peb}) \cdot S_{cgb} - M_{DC1} \cdot \left( \frac{S_{cgb}}{S_{gb}} - 1 \right)$$

$$= (1.08 + 4.98) \cdot 21,664 - (4642 \cdot 12) \cdot \left( \frac{21,664}{14,673} - 1 \right) = 104,743 \text{ kip-in}$$

$$= 8729 \text{ kip-ft}$$

$$1.2 \cdot M_{cr} = 1.2 \cdot 8729 = 10,475 < 16,013 \text{ kip-ft provided} \quad \text{OK}$$

**E. Design  
Reinforcement for  
Shear**

[5.8]

**1. Vertical Shear Design  
Determine  $d_v$  and Critical Section for Shear**

Begin by determining the effective shear depth  $d_v$  at the critical section for shear.

$$d_v = d_p - \frac{a}{2}$$

The effective shear depth is no less than:

$$d_v \geq 0.72 \cdot h = 0.72 \cdot (63 + 1.5 + 8.5) = 52.6 \text{ in}$$

Assume  $d_v = 52.6$  inches at critical section location of  $d_v$  from face of support. The internal face is assumed to be at the inside edge of the 15 inch sole plate. Then the critical section will be at least 67.6 inches ( $52.6 + 15$ ) or 5.63 feet away from the end of the beam. Find the centroid of the prestressing strands at this location:

The centroid of the prestressing strands is at:

$$y_{\text{str}@d_v} = y_{\text{end}} - \left( \frac{5.63}{0.40 \cdot \text{span} + \frac{L_{\text{soleplate}}}{2}} \right) \cdot (y_{\text{end}} - y_{\text{drapel}})$$

$$= 17.26 - \left( \frac{5.63}{0.40 \cdot 137 + \frac{15}{2 \cdot 12}} \right) \cdot (17.26 - 5.59) = 16.1 \text{ in}$$

With this approximation to the strand centroid,  $d_p$  can be computed:

$$d_p = h - y_{\text{str}@d_v} = (63 + 1.5 + 8.5) - 16.1 = 56.9 \text{ in}$$

From the flexural strength computations,  $a = 8.27$  inches at midspan. The value of "a" varies slightly along the beam length, but the value at midspan is close enough for design purposes.

$$d_v = d_p - \frac{a}{2} = 56.9 - \frac{8.27}{2} = 52.8 \text{ in}$$

But the effective shear depth  $d_v$  need not be less than

$$d_v \geq 0.72 \cdot h = 52.6 \text{ in}$$

or

$$d_v \geq 0.9 d_e = 0.9 d_p = 0.9 (56.9) = 51.2 \text{ in}$$

Therefore take  $d_v = 52.8$  inches, which is sufficiently close to the original assumption of  $d_v = 52.6$  in

Then  $x_{\text{critv}} = 7.5 + d_v = 7.5 + 52.8 = 60.3 \text{ in} = 5.03 \text{ ft}$  from centerline of bearing

#### Check Maximum Factored Shear Limit

From Table 5.7.2.4 the Strength I design shear at 5.03 ft is

$$V_u = 376 \text{ kips}$$

The amount of force carried by the draped strands at their effective prestress level is:

$$P_{14d} = 14 \cdot 0.217 \cdot 150.6 = 457.5 \text{ kips}$$

The inclination of the draped strands is:

$$\phi = \arctan \left[ \frac{(54 - 9)/12}{55.43} \right] = 3.87 \text{ degrees}$$

The vertical prestress component is:

$$V_p = P_{14d} \cdot \sin(\phi) = 457.5 \cdot \sin(3.87) = 30.9 \text{ kips}$$

#### [5.8.3.2]

The superstructure is supported by a parapet type abutment. Therefore, the nominal shear capacity of the section is limited to:

$$V_n = 0.18 \cdot f'_c \cdot d_v \cdot b_v + V_p = 0.18 \cdot 8.5 \cdot 52.8 \cdot 6.5 + 30.9 = 556 \text{ kips}$$

The maximum design shear the section can have is:

$$\phi_v \cdot V_n = 0.90 \cdot 556 = 500 \text{ kips} > 376 \text{ kips}$$

OK

**Determine Longitudinal Strain  $\epsilon_x$** 

Assume that minimum transverse reinforcement will be provided in the cross section.

First, determine  $A_{ps}$ . Note that  $A_{ps}$  computed here is different than the  $A_{ps}$  computed earlier. This  $A_{ps}$  includes only the area of prestressing steel found on the flexural tension side of the member. Near the end of the beam,  $A_{ps}$  must also be reduced for development.

Development length  $\ell_d$  is:

$$\begin{aligned}\ell_d &= K \left( f_{ps} - \frac{2}{3} f_{pe} \right) d_b \\ &= 1.6 \left[ 259.1 - \frac{2}{3} (150.6) \right] (0.6) = 152.4 \text{ in}\end{aligned}$$

Transfer length  $\ell_{tr}$  is:

$$\ell_{tr} = 60 \cdot d_b = 60(0.6) = 36.0 \text{ in}$$

At the critical section  $x_{critve} = (60.3 + 15/2) = 67.8$  in from the beam end, the strand development fraction is:

$$\begin{aligned}F_{dev} &= \frac{f_{pe}}{f_{pu}} + \frac{d_{critv} - \ell_{tr}}{\ell_d - \ell_{tr}} \left( 1 - \frac{f_{pe}}{f_{pu}} \right) \\ &= \frac{150.6}{270} + \frac{67.8 - 36.0}{152.4 - 36.0} \left( 1 - \frac{150.6}{270} \right) = 0.68\end{aligned}$$

The flexural tension side of the member is defined as:

$$\frac{h_{comp}}{2} = \frac{63}{2} = 31.5 \text{ in}$$

At  $x_{critve}$  none of the draped strands fall on the flexural tension side. Therefore,  $A_{ps} = (\# \text{ straightstr.})(\text{strandarea})(F_{dev})$

$$= (40)(0.217)(0.68) = 5.90 \text{ in}^2$$

**[5.8.3.4.2]**

Use equation 5.8.3.4.2-4 to compute the strain:

$$\epsilon_s = \left[ \frac{\frac{M_u}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps} \cdot f_{po}}{E_s \cdot A_s + E_p \cdot A_{ps}} \right]$$

$$= \left[ \frac{\frac{1713 \cdot 12}{52.8} + |376 - 30.9| - (5.90 \cdot 0.70 \cdot 270)}{28,500 \cdot 5.90} \right] = -0.002$$

Because the value is negative, the strain will be recalculated using an additional concrete term:

From Figure 5.4.6.1 of this manual,  $A_c = 486 \text{ in}^2$

For  $f'_c = 8.5 \text{ ksi}$ ,  $E_c = 1265 \cdot \sqrt{8.5} + 1000 = 4688 \text{ ksi}$

$$\varepsilon_s = \left[ \frac{\frac{M_u}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps} \cdot f_{po}}{E_c \cdot A_c + E_s \cdot A_s + E_p \cdot A_{ps}} \right]$$

$$= \left[ \frac{\frac{1713 \cdot 12}{52.8} + |376 - 30.9| - (5.90 \cdot 0.70 \cdot 270)}{28,500 \cdot 5.90 + 4688 \cdot 486} \right] = -0.00016$$

Computed strain limits:

$$-0.0004 < -0.00016 < 0.006$$

OK

Compute the tensile stress factor  $\beta$  using equation 5.8.3.4.2-1

$$\beta = \frac{4.8}{1 + 750 \cdot \varepsilon_s} = \frac{4.8}{1 + 750 \cdot (-0.00016)} = 5.45$$

Compute the angle  $\theta$  using equation 5.8.3.4.2-3

$$\theta = 29 + 3500\varepsilon_s = 29 + 3500 \cdot (-0.00016) = 28.44 \text{ degrees}$$

Compute the concrete contribution:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 5.45 \cdot \sqrt{8.5} \cdot 6.5 \cdot 52.8 = 172.3 \text{ kips}$$

The required steel contribution is:

$$V_s = V_n - V_c - V_p = \frac{V_u}{\phi_v} - V_c - V_p = \frac{376}{0.90} - 172.3 - 30.9 = 214.6 \text{ kips}$$

Find the required spacing of double leg #4 stirrups:

$$s = \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{V_s} = \frac{2 \cdot 0.20 \cdot 60 \cdot 52.8 \cdot \cot(28.44)}{214.6} = 10.90 \text{ in}$$

Try double leg stirrups at a 10 inch spacing at the end of the beam.

$$A_v = \frac{0.4 \cdot 12}{10} = 0.48 \text{ in}^2 / \text{ft} \quad V_s = 234.0 \text{ kips}$$

**[5.8.2.5]**

Check that the minimum transverse reinforcement requirement is satisfied:

$$\begin{aligned} \frac{A_{vmin}}{s} &= 0.0316 \sqrt{f'_c} \cdot \frac{b_v}{f_y} \\ &= 0.0316 \sqrt{8.5} \cdot \frac{6.5}{60} \cdot 12 = 0.12 \frac{\text{in}^2}{\text{ft}} < 0.48 \frac{\text{in}^2}{\text{ft}} \quad \text{OK} \end{aligned}$$

**[5.8.2.7]**

Check maximum stirrup spacing:

$$\begin{aligned} V_{spac} &= 0.125 \cdot f'_c \cdot b_v \cdot d_v \\ &= 0.125 \cdot 8.5 \cdot 6.5 \cdot 52.8 = 364.7 \text{ kips} < 376 \text{ kips} \end{aligned}$$

Then the maximum spacing is the smaller of:

$$s_{max} = 0.4 \cdot d_v = 0.4 \cdot 52.8 = 21.1 \text{ in}$$

$$\text{or} \quad s_{max} = 12 \text{ in} \quad \text{GOVERNS}$$

$$s_{max} = 12 \text{ in} > 10 \text{ in} \quad \text{OK}$$

Therefore use double leg #13 stirrups at 10 inch spacing. Other sections are investigated similarly.

**[5.8.4]**

## 2. Interface Shear Transfer

Top flange width  $b_v = 34 \text{ in}$

The Strength I vertical shear at the critical shear section due to all loads is:

$$V_u = 376$$

Interface shear force is:

$$V_h = \frac{V_u}{d_e} = \frac{V_u}{d_v} = \frac{376}{52.8} \cdot \frac{12 \text{ in}}{\text{ft}} = 85.5 \text{ kip/ft}$$

Required nominal interface design shear is:

$$V_{nreq} = \frac{V_h}{\phi_v} = \frac{85.5}{0.90} = 95.0 \text{ kip/ft}$$

The interface area per 1 foot length of beam is:

$$A_{cv} = 34 \cdot 12 = 408.0 \text{ in}^2/\text{ft}$$

**[5.8.4.3]**

A note on Bridge Details II Fig. 5-397.509 requires the top flanges of the beam to be roughened. Then:

$$c = 0.28 \text{ ksi} \quad \mu = 1.0 \quad K_1 = 0.3 \quad K_2 = 1.8 \text{ ksi}$$

The upper limits on nominal interface shear are:

$$K_1 \cdot f'_c \cdot A_{cv} = 0.3 \cdot 4 \cdot 408.0 = 489.6 \text{ kip/ft} > 95.0 \text{ kip/ft} \quad \underline{\text{OK}}$$

and

$$K_2 \cdot A_{cv} = 1.8 \cdot 408.0 = 734.4 \text{ kip/ft} > 95.0 \text{ kip/ft} \quad \underline{\text{OK}}$$

The nominal interface shear resistance is:

$$V_n = c A_{cv} + \mu (A_{vf} \cdot f_y + P_c)$$

$$P_c = 0.0 \text{ kip}$$

Substitute and solve for required interface shear steel:

$$A_{vf} = \frac{V_{nreq} - c A_{cv}}{\mu \cdot f_y} = \frac{95.0 - 0.28(408.0)}{1.0 \cdot 60} = -0.32 \text{ in}^2/\text{ft}$$

Calculated value is negative, so  $A_{vf}$  is taken as zero

**[ 5.8.4.4]**

Check minimum interface shear requirements:

$$A_{vfmin} = \frac{0.05 \cdot b_v}{f_y} = \frac{0.05 \cdot 34}{60} = 0.028 \text{ in}^2/\text{in} = 0.34 \text{ in}^2/\text{ft}$$

The minimum requirement may be waived for girder-slab interfaces with the surface roughened to an amplitude of 0.25 in if the factored interface shear stress is less than 0.210 ksi.

$$v_{ui} = \frac{V_{ui}}{b_v d_v} = \frac{376.0}{34 \cdot 52.8} = 0.209 \text{ ksi} < 0.210 \text{ ksi}$$

No additional reinforcement is required for interface shear. Other sections are investigated similarly.

**[5.8.3.5]****3. Minimum Longitudinal Reinforcement Requirement**

The longitudinal reinforcement must be checked to ensure it is adequate to carry the tension caused by shear. The amount of strand development must be considered near the end of the beam. There are 2 cases to be checked:

Case 1: From the inside edge of bearing at the end supports out to a distance  $d_v$ , the following must be satisfied:

$$A_{ps} \cdot f_{ps} \geq \left( \frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p \right) \cdot \cot \theta$$

A crack starting at the inside edge of the bearing sole plate will cross the center of gravity of the straight strands at:

$$x_{\text{crack}} = L_{\text{soleplate}} + y_{\text{str}} \cdot \cot(\theta) = 15 + 4.40 \cdot \cot(28.44) = 23.1 \text{ in}$$

The transfer length for 0.6" strands is:  $\ell_{tr} = 36.0 \text{ in}$

Interpolate to find the tensile capacity of the straight strands at the crack:

$$T_r = f_{pe} \cdot A_{ps} \cdot \frac{x_{\text{crack}}}{\ell_{tr}} = 150.6 \cdot 40 \cdot 0.217 \cdot \frac{23.1}{36} = 838.8 \text{ kips}$$

The force to carry is:

$$\begin{aligned} T &= \left( \frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p \right) \cdot \cot(\theta) \\ &= \left( \frac{376}{0.90} - 0.5 \cdot 234.0 - 30.9 \right) \cdot \cot(28.44) \\ &= 498.3 \text{ kips} < 838.8 \text{ kips} \end{aligned}$$

OK

Case 2: At  $d_v$ , the following must be satisfied:

$$A_{ps} \cdot f_{ps} \geq \frac{M_u}{\phi d_v} + \left( \frac{V_u}{\phi} - 0.5 \cdot V_s - V_p \right) \cdot \cot \theta$$

Based on the fraction of strands developed as calculated earlier:

$$A_{ps} = 40 \cdot 0.217 \cdot 0.68 = 5.90 \text{ in}^2$$

Following steps shown earlier,  $f_{ps}$  at  $d_v$  from the inside edge of bearing is:

$$f_{ps} = 257.1 \text{ ksi}$$

$$\text{Then } T_r = A_{ps} \cdot f_{ps} = 5.90 \cdot 257.1 = 1516.9 \text{ kips}$$

$$T = \frac{M_u}{\phi d_v} + \left( \frac{V_u}{\phi_v} - 0.5 \cdot V_s - V_p \right) \cdot \cot(\theta) = \frac{1713 \cdot 12}{1.0 \cdot 52.8} + 498.3$$

$$T = 887.6 \text{ kips} < 1516.9 \text{ kips}$$

OK

**F. Design**  
**Pretensioned**  
**Anchorage Zone**  
**Reinforcement**  
**[5.10.10.1]**

**Splitting Reinforcement**

To prevent cracking in the beam end due to the transfer of the prestressing force from the strands to the concrete, splitting steel needs to be provided in the anchorage zone.

Use a load factor of 1.0 and lateral force component of 4% to determine the required amount of steel.

The factored design bursting force is:

$$P_b = 1.0 \cdot 0.04 \cdot P_i = 1.0 \cdot 0.04 \cdot 2080 = 83.2 \text{ kips}$$

The amount of resisting reinforcement is determined using a steel stress  $f_s$  of 20 ksi:

$$A_s = \frac{P_b}{f_s} = \frac{83.2}{20} = 4.16 \text{ in}^2$$

This steel should be located at the end of the beam within a distance of:

$$\frac{h}{4} = \frac{63}{4} = 15.75 \text{ in}$$

The number of #5 double legged stirrups necessary to provide this area is:

$$\frac{A_s}{2 \cdot A_b} = \frac{4.16}{2 \cdot 0.31} = 6.7$$

The first set of stirrups is located 2 inches from the end of the beam. Provide seven sets of #5 stirrups spaced at 2 1/2 inch centers.

$$x_{\text{splitting}} = 2 + 6 \cdot 2.5 = 17 \text{ in} > 15.75 \text{ in}$$

Although the splitting reinforcement does not fit within  $h/4$ , #5 bars are the largest allowed and 2.5 inches is the tightest spacing allowed. This is OK per MnDOT practice.

[5.10.10.2]

**Confinement Reinforcement**

Reinforcement is required at the ends of the beam to confine the prestressing steel in the bottom flange. G403E and G507E bars (see Figure 5.7.2.5) will be placed at a maximum spacing of 6 inches out to 1.5d from the ends of the beam. For simplicity in detailing and ease of tying the reinforcement, space the vertical shear reinforcement with the confinement reinforcement in this area.

$$1.5d = 1.5(63) = 94.5 \text{ in}$$

**G. Determine Camber and Deflection**

[2.5.2.6.2]

[3.6.1.3.2]

[5.7.3.6.2]

**Camber Due to Prestressing and Dead Load Deflection**

Using the PCI handbook (Figure 4.10.13 of the 3<sup>rd</sup> Edition), the camber due to prestress can be found. The centroid of the prestressing has an eccentricity  $e_{mid}$  of 23.21 inches at midspan. At the end of the beams the eccentricity  $e_e$  is 12.17 inches. E is the initial concrete modulus (4394 ksi),  $P_o$  equals the prestress force just after transfer (2080 kips). The drap points are at 0.4 of the span. The span length is 137.0 feet. Using the equation for the two point depressed strand pattern:

$$e' = e_{mid} - e_e = 23.21 - 12.17 = 11.04 \text{ in}$$

$$\begin{aligned} \Delta_{ps} &= \frac{P_o e_e L^2}{8EI} + \frac{P_o e'}{EI} \left( \frac{L^2}{8} - \frac{a^2}{6} \right) \\ &= \frac{2080(12.17)(137 \cdot 12)^2}{8(4394)(422,570)} + \frac{2080(11.04)}{4394(422,570)} \left[ \frac{(137 \cdot 12)^2}{8} - \frac{(0.4 \cdot 137 \cdot 12)^2}{6} \right] \\ &= 7.89 \text{ in} \end{aligned}$$

Downward deflection due to selfweight

$$\Delta_{sw} = \frac{5 \cdot W \cdot L^4}{384 \cdot E \cdot I} = \frac{5 \cdot \frac{0.869}{12} (137 \cdot 12)^4}{384 \cdot 4394 \cdot 422,570} = -3.71 \text{ in}$$

$$\text{Camber at release } \Delta_{rel} = \Delta_{ps} - \Delta_{sw} = 7.89 - 3.71 = 4.18 \text{ in}$$

To estimate camber at the time of erection the deflection components are multiplied by standard MnDOT multipliers. They are:

Release to Erection Multipliers:

Prestress = 1.4

Selfweight = 1.4

Camber and selfweight deflection values at erection are:

Prestress:  $1.4 \cdot 7.89 = 11.05 \text{ in}$

Selfweight:	$1.4 \cdot (-3.71) = -5.19$ in
Diaphragm DL:	-0.04 in
Deck and stool DL:	-4.41 in
Parapet:	-0.24 in

The values to be placed in the camber diagram on the beam plan sheet are arrived at by combining the values above.

"Erection Camber" =  $11.05 - 5.19 - 0.04 = 5.82$  in    say 5 7/8 in

"Est. Dead Load Deflection" =  $4.41 + 0.24 = 4.65$  in    say 4 5/8 in

"Est. Residual Camber" =  $5 \frac{7}{8} - 4 \frac{5}{8} = 1 \frac{1}{4}$  in

**Live Load Deflection**

The deflection of the bridge is checked when subjected to live load and compared against the limiting values of  $L/800$  for vehicle only bridges and  $L/1000$  for bridges with bicycle or pedestrian traffic.

Deflection due to lane load is:

$$\Delta_{lane} = \left( \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I} \right) = \left[ \frac{5 \cdot \frac{0.64}{12} \cdot (137 \cdot 12)^4}{384 \cdot 4688 \cdot 1,034,168} \right] = 1.05 \text{ in}$$

Deflection due to a truck with dynamic load allowance is found using hand computations or computer tools to be:

$$\Delta_{truck} = 1.74 \text{ in}$$

Two deflections are computed and compared to the limiting values; that of the truck alone and that of the lane load plus 25% of the truck. Both deflections need to be adjusted with the distribution factor for deflection.

$$\Delta_1 = DF_{\Delta} \cdot \Delta_{truck} = 0.567 \cdot 1.74 = 0.99 \text{ in}$$

$$\Delta_2 = DF_{\Delta} \cdot (\Delta_{lane} + 0.25 \cdot \Delta_{truck}) = 0.567 \cdot (1.05 + 0.25 \cdot 1.74) = 0.84 \text{ in}$$

There is no bicycle or pedestrian traffic on the bridge, so the deflection limit is:

$$\frac{L}{800} = \frac{137 \cdot 12}{800} = 2.1 \text{ in} \gg \text{ than } \Delta_1 \text{ or } \Delta_2 \quad \underline{OK}$$

**H. Detailing Items**

Approximate weight of each beam is:

$$L \cdot A \cdot \gamma = 138.25 \cdot \frac{807}{144} \cdot 0.155 \cdot \frac{1}{2} = 60.0 \text{ tons}$$

Initial prestress force at jacking is:

$$54 \cdot 0.217 \cdot 0.75 \cdot 270 = 2373 \text{ kips}$$

Figure 5.7.2.5 shows the detailed beam sheet for the bridge.

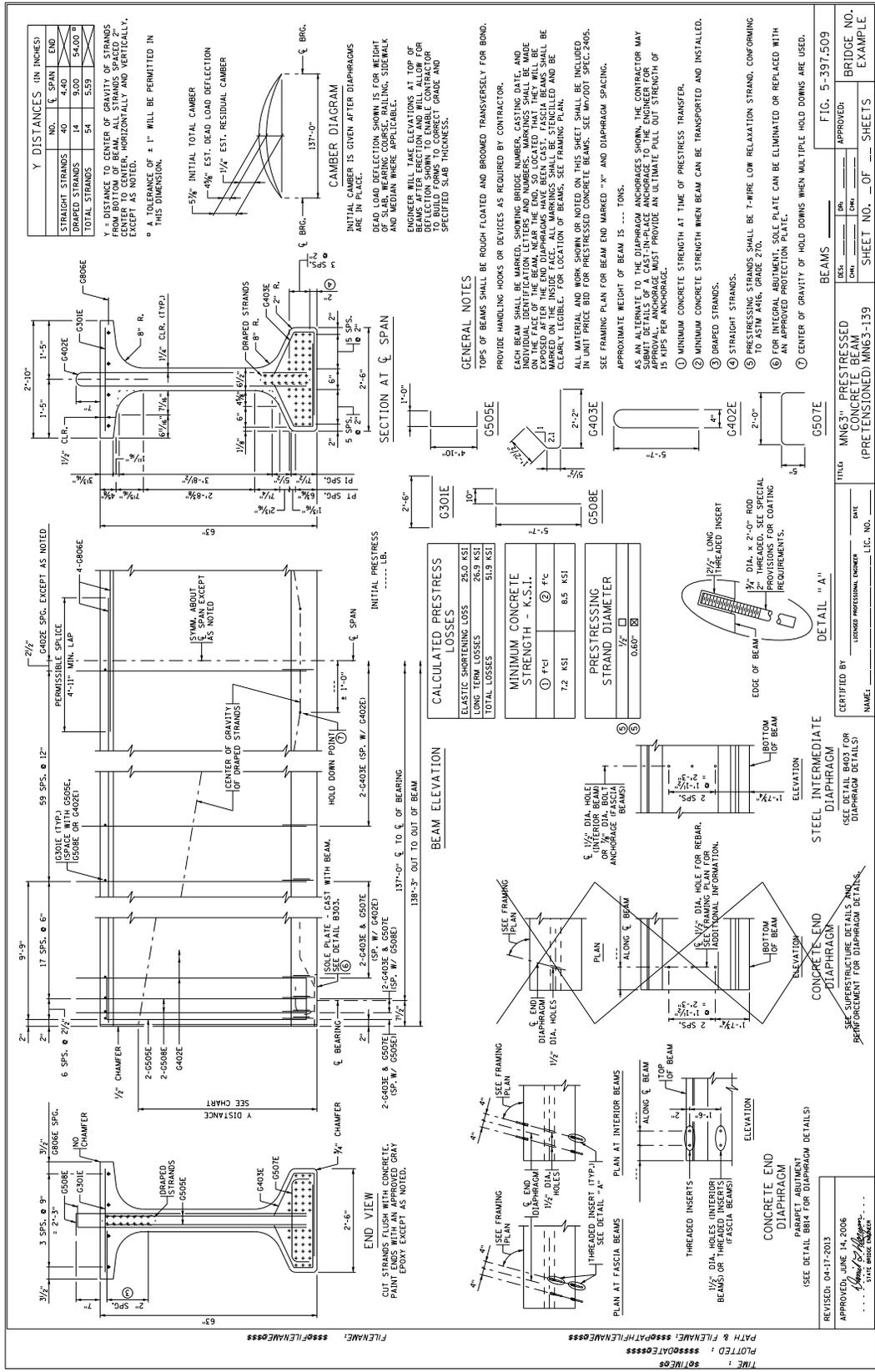


Figure 7.2.5

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**5.7.3 Three-Span Haunched Post-Tensioned Concrete Slab Design Example**

This example illustrates the design of a haunched post-tensioned concrete slab bridge. The three continuous spans are 55'-0", 70'-0", and 55'-0" in length. The roadway width is 44'-0" with MnDOT Type F barrier railings for a total out-to-out width of 47'-4". A plan view and typical sections of the bridge are shown in Figures 5.7.3.1 and 5.7.3.2.

After computing the dead and live loads, a preliminary tendon profile is developed. Prestress losses for the preliminary layout are computed for anchor set, friction, elastic shortening, creep, shrinkage, and relaxation. Subsequently, the load combinations are assembled (with the secondary post-tensioning force effects included). Flexural and shear strength checks are performed, after which deflection and camber calculations are assembled. Lastly, the design of the anchorage zone is performed.

Single ended jacking is assumed for the design. The construction documents will require that the jacked end and the dead ends alternate. With the tendons stressed at alternating ends, the results for the friction losses and anchor set losses for tendons stressed at opposite ends will be averaged to obtain losses for a "typical" tendon.

The following material and design parameters are used in this example:

**A. Material and Design Parameters**

**Table 5.7.3.1 Design Parameters**

Material	Parameter	Value	
Concrete	Compressive Strength at Transfer, $f'_{ci}$	4.5 ksi	
	Compressive Strength at 28 days, $f'_c$	5.0 ksi	
	Modulus of Elasticity at Transfer, $E_{ci}$	3865 ksi	
	Modulus of Elasticity at 28 days, $E_c$	4074 ksi	
Steel Reinforcement	Prestressing	7-wire Strand	0.60 in dia., low-lax
		Area of one Strand	0.217 in <sup>2</sup>
		Tensile Strength, $f_{pu}$	270.0 ksi
		Yield Strength, $f_{py}$	243.0 ksi
		Modulus of Elasticity, $E_p$	28,500 ksi
	Rebar	Yield Strength, $F_y$	60 ksi
		Modulus of Elasticity, $E_s$	29,000 ksi

[5.4.2.4]

[Table 5.4.4.1-1]

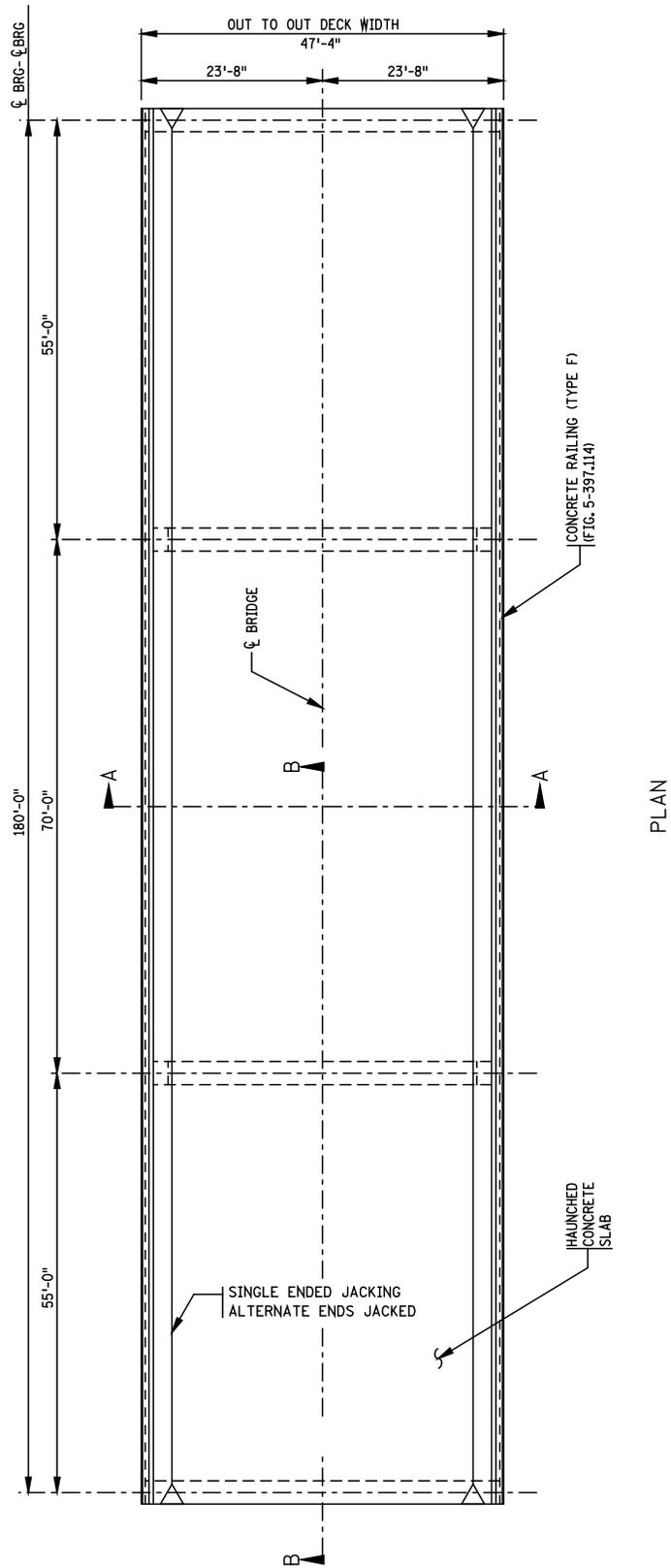
[5.4.4.2]

[5.4.3.2]

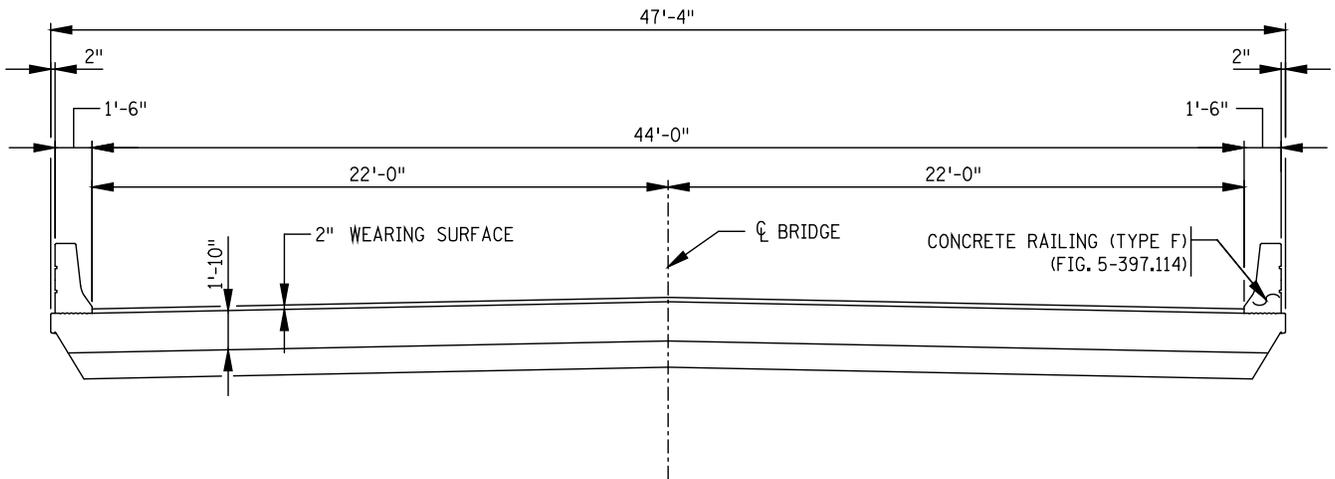
**Additional Dead Loads**

Future Wearing Surface = 0.020 ksf

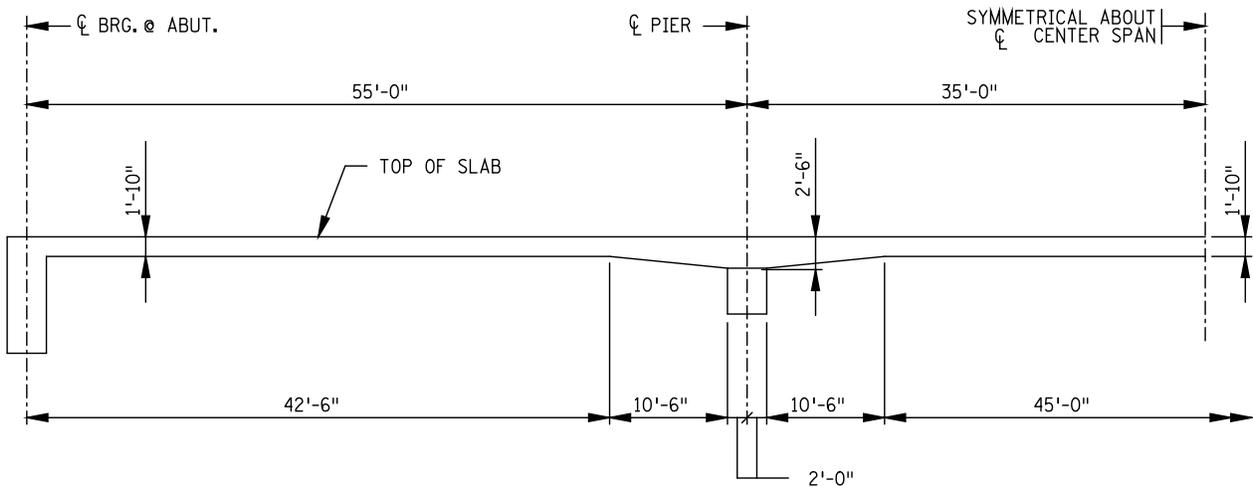
Type F Barriers,  $w = 0.439$  kip/ft/barrier



**Figure 5.7.3.1**  
**Bridge Layout**



SECTION A-A



SECTION B-B

**Figure 5.7.3.2**  
**Transverse and Longitudinal Sections**

**B. Haunch Length  
and Minimum  
Recommended Slab  
Depth**

MnDOT's standard design practice is to use linear haunches, with a haunch length of 15% of the length of the longest span in the continuous system.

Haunch Length:

$$0.15 \cdot 70 = 10.5 \text{ ft} \quad \text{Use } \underline{10.5 \text{ ft}}$$

**[2.5.2.6.3]**

The depth of the slab at midspan can be estimated with:

$$0.95 \cdot (0.027 \cdot L) = 0.95 \cdot (0.027 \cdot 70) = 1.80 \text{ ft} \quad \text{Use } \underline{1.83 \text{ ft}}$$

A trial depth of the slab at the piers can be found with:

$$\frac{4}{3} \cdot (\text{midspan slab depth}) = \frac{4}{3} \cdot (1.80) = 2.39 \text{ ft} \quad \text{Use } \underline{2.50 \text{ ft}}$$

**C. Live Load Strip  
Widths and  
Distribution Factors**  
**[4.6.2.3]**  
**[3.6.1.1.1]**

The equations in the LRFD Specifications are arranged to determine the width of slab that resists a particular live load. To simplify the design process (which is based on a 1 foot wide design strip) the resultant widths are rearranged to determine the fraction of lane load carried by a 1 foot wide strip of slab.

**Distribution Factor for Flexure – One Lane Loaded**

The equation used to find the width of slab resisting one lane of live loading is:

$$E = 10.0 + 5.0 \cdot \sqrt{L_1 \cdot W_1}$$

Where:

$L_1$  is the modified span length (the smaller of the actual span length and 60 feet)

$W_1$  is the modified bridge width (the smaller of the actual width and 30 feet)

Substituting in values for the side and main spans produces:

55 ft Spans:

$$E_s = 10.0 + 5.0 \cdot \sqrt{55 \cdot 30} = 213.1 \text{ in/lane}$$

$$\frac{1}{E_s} = \left( \frac{1}{213.1} \right) \cdot \left( \frac{12}{1} \right) = 0.056 \text{ lanes/ft}$$

70 ft Span:

$$E_s = 10.0 + 5.0 \cdot \sqrt{60 \cdot 30} = 222.1 \text{ in/lane}$$

$$\frac{1}{E_s} = \left( \frac{1}{222.1} \right) \cdot \left( \frac{12}{1} \right) = 0.054 \text{ lanes/ft}$$

### Distribution Factor for Flexure – Multiple Lanes Loaded

A similar procedure is used to determine the width of slab that carries multiple lanes of live load. The general equation is:

$$E = 84.0 + 1.44 \cdot \sqrt{L_1 \cdot W_1} \leq \frac{12.0 \cdot W}{N_L}$$

Where:

$L_1$  is the modified span length (smaller of the span length and 60 ft)

$W_1$  is the modified bridge width (smaller of the bridge width and 60 ft)

$W$  is the physical edge-to-edge width of the bridge (47.33 ft)

$N_L$  is the number of design lanes:

$$N_L = \frac{44}{12} = 3.7 \quad \text{Use 3}$$

Substituting values into the equations for the side and main spans produces:

55 ft Spans:

$$E_m = 84.0 + 1.44 \cdot \sqrt{55 \cdot 47.33} = 157.5 \leq \frac{12.0 \cdot 47.33}{3} = 189.3 \text{ in/lane}$$

$$\frac{1}{E_m} = \left( \frac{1}{157.5} \right) \cdot \left( \frac{12}{1} \right) = 0.076 \text{ lanes/ft}$$

70 ft Span:

$$E_m = 84.0 + 1.44 \cdot \sqrt{60 \cdot 47.33} = 160.7 \leq \frac{12.0 \cdot 47.33}{3} = 189.3 \text{ in/lane}$$

$$\frac{1}{E_m} = \left( \frac{1}{160.7} \right) \cdot \left( \frac{12}{1} \right) = 0.075 \text{ lanes/ft}$$

### Distribution Factor for Shear

The shear check is performed with a single distribution factor where all design lanes are loaded and the entire slab is assumed to participate in carrying the load.

$$E_v = (\# \text{ of lanes}) \cdot \left( \frac{\text{MPF}}{\text{deck width}} \right) = 3 \cdot \left( \frac{0.85}{47.33} \right) = 0.054 \text{ lanes/ft}$$

**[2.5.2.6.2]****Distribution Factor for Deflection**

Deflection computations should be based on the same distribution factor calculated for shear forces:

$$E_{\Delta} = E_v = 0.054 \text{ lanes/ft}$$

**D. Edge Beam  
Width and  
Distribution Factor  
[4.6.2.1.4]**

The exterior strip is assumed to carry one wheel line and a tributary portion of lane live load.

Check if the equivalent strip is less than the maximum width of 72 inches.

$$E = (\text{edge of slab to inside face of barrier}) + 12.0 + \frac{(\text{strip width})}{2}$$

$$E = 20 + 12 + \frac{157.4}{2} = 110.7 \text{ in} > 72.0 \text{ in} \quad \text{Use } 72.0 \text{ in}$$

Compute the distribution factor associated with one truck wheel line:

$$\begin{aligned} \text{LLDF}_{\text{EXTT}} &= \left[ \frac{1 \text{ wheel line} \cdot (\text{MPF})}{(2 \text{ wheel lines/lane}) \cdot (E/12)} \right] \\ &= \left[ \frac{1 \cdot (1.2)}{2 \cdot (72/12)} \right] = 0.100 \text{ lanes/ft} \end{aligned}$$

Compute the distribution factor associated with lane load on a 72 inch wide exterior strip. Subtract the gutter line to edge of deck distance to obtain the deck width loaded:

$$\begin{aligned} \text{LLDF}_{\text{EXTL}} &= \left[ \frac{\left( \frac{\text{deck width loaded}}{10 \text{ ft. load width}} \right) \cdot \text{MPF}}{(\text{exterior strip width})} \right] \\ \text{LLDF}_{\text{EXTL}} &= \left[ \frac{\left( \frac{72/12 - 20/12}{10} \right) \cdot 1.2}{72/12} \right] \text{ lanes/ft} \end{aligned}$$

For simplicity, the larger value (0.100 lanes/ft) is used for both load types when assembling design forces for the exterior strip.

Table 5.7.3.2 summarizes the distribution factors for the different force components.

**Table 5.7.3.2  
Distribution Factor Summary**

Force Component	Width	Type of Loading	Span (ft)	Distribution Factor (lanes/ft)
Flexure	Interior Strip	One Lane	55	0.056
			70	0.054
		Multiple Lanes	55	0.076
			70	0.075
	Exterior Strip	One Lane	55 & 70	0.100
Shear	Slab Width	Multiple Lanes	55 & 70	0.054
Deflections	Slab Width	Multiple Lanes	55 & 70	0.054

The following load modifiers will be used for this example:

		Strength	Service	Fatigue
Ductility	$\eta_D$	1.0	1.0	1.0
Redundancy	$\eta_R$	1.0	1.0	1.0
Importance	$\eta_I$	1.0	n/a	n/a
	$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.0	1.0	1.0

**E. Load Combinations, Load Factors, and Load Modifiers**  
**[3.4.1]**  
**[1.3.3-1.3.5]**

The load combinations considered for the design example are identified below:

STRENGTH I – Used to ensure adequate strength under normal vehicular use.

$$U = 1.0 \cdot [1.25 \cdot DC + 1.25 \cdot DW + 1.75 \cdot (LL + IM)]$$

SERVICE I – Used for compression checks in prestressed concrete.

$$U = 1.0 \cdot (DC + DW) + 1.0 \cdot (LL + IM)$$

SERVICE III – Used for tension checks in prestressed concrete for crack control purposes.

$$U = 1.0 \cdot (DC + DW) + 0.8 \cdot (LL + IM)$$

**[5.5.3.1]**

FATIGUE – No fatigue check is necessary for fully prestressed sections.

**F. Live Loads**  
**[3.6.1]**

The HL-93 live load components used for this example are:

Design Truck  
 Design Lane  
 Design Tandem  
 Truck Train

The live load components are combined in the following manner:

Design Truck + Design Lane  
 Design Tandem + Design Lane  
 0.90 (Truck Train + Design Lane) (Neg. Moment Regions)

**[3.6.2]**

**Dynamic Load Allowance**

The dynamic load allowance, (IM) for truck and tandem live loads is 33% for all applicable limit states and load combinations.

**G. Dead Loads**

**Interior Strip (1'-0" Width)**

The 2 inch wearing course is included in the slab depth (h) used to determine the dead loads ( $w_{DC}$ ). It is not considered part of the structural section resisting loads.

$$w_{DC} = (\text{width}) \cdot w_c \cdot h + \left( \frac{2 \cdot w_{\text{barrier}}}{\text{deck width}} \right)$$

$$= (1.0) \cdot 0.150 \cdot h + \left( \frac{2 \cdot 0.439}{47.33} \right) = 0.150 \cdot h + 0.019 \text{ kip/ft}$$

For design simplicity the dead load associated with the future wearing surface (0.020 ksf) is combined with the other DC loads.

$$w_{DC} = 0.150 \cdot h + 0.019 + 0.020 = 0.150 \cdot h + 0.039 \text{ kip/ft}$$

**Edge Strip (1'-0" Width)**

For the design of the edge strip, it is conservatively assumed that the dead load of one barrier is carried by each edge strip.

$$w_{DC} = 0.150 \cdot h + \left( \frac{0.439}{6.0} \right) = 0.150 \cdot h + 0.073 \text{ kip/ft}$$

The future wearing surface load is:

$$w_{DW} = 0.120 \cdot \left( \frac{6.0 - 1.67}{6.0} \right) = 0.014 \text{ kip/ft}$$

The combined dead load is:

$$w_{DC} = 0.150 \cdot h + 0.073 + w_{DW} = 0.150 \cdot h + 0.087 \text{ kip/ft}$$

***H. Structural  
Analysis Model and  
Resultant Loads***

The dead and live loads were applied to a continuous beam model with gross section properties. Nonprismatic properties were used to account for the presence of the linear haunches near the piers. The results of the analysis are presented in Tables 5.7.3.3 and 5.7.3.4.

**Table 5.7.3.3**  
**Moment Load Components (kip-ft)**

Span Point	Dead Load (per ft)		Live Load (per lane)*						
	Interior Strip M <sub>DC</sub>	Exterior Strip M <sub>DC</sub>	Lane		Truck		Tandem		Truck Train
			Max.	Min.	Max.	Min.	Max.	Min.	Min.
1.0	0	0	0	0	0	0	0	0	-
1.1	31	35	78	-20	263	-47	227	-36	-
1.2	51	59	136	-39	433	-94	384	-71	-
1.3	62	71	175	-59	519	-140	475	-107	-
1.4	62	71	195	-78	552	-187	507	-142	-
1.5	52	59	196	-98	534	-234	492	-178	-
1.6	31	36	177	-118	476	-281	437	-213	-
1.7	1	1	138	-137	359	-327	345	-249	-286
1.8	-40	-46	83	-159	204	-374	228	-284	-380
1.9	-92	-105	52	-225	116	-421	97	-320	-516
2.0	-155	-177	46	-335	129	-468	102	-355	-698
2.1	-78	-89	42	-190	77	-335	118	-265	-483
2.2	-20	-23	71	-109	246	-284	268	-224	-303
2.3	22	25	133	-93	416	-232	392	-183	-
2.4	47	53	181	-93	520	-180	472	-143	-
2.5	55	63	196	-93	545	-129	496	-102	-

\* Values do not include dynamic load allowance.

**Table 5.7.3.4**  
**Shear Load Components (kips)**

Span Point	Dead Load (per ft)	Live Load (per lane)*					
		Lane		Truck		Tandem	
		Max.	Min.	Max.	Min.	Max.	Min.
1.0	6.5	15.9	-3.6	56.7	-8.5	47.7	-6.5
1.1	4.7	12.6	-3.8	47.8	-8.5	41.2	-6.5
1.2	2.8	9.8	-4.5	39.4	-8.5	34.9	-10.5
1.3	0.9	7.4	-5.6	31.4	-14.1	28.8	-16.8
1.4	-0.9	5.4	-7.1	24.1	-22.0	23.0	-22.8
1.5	-2.8	3.8	-9.1	17.4	-29.6	17.7	-28.5
1.6	-4.6	2.6	-11.4	11.6	-37.6	12.8	-33.7
1.7	-6.5	1.8	-14.1	6.5	-45.2	8.5	-38.5
1.8	-8.3	1.2	-17.0	2.3	-52.1	4.9	-42.6
1.9	-10.4	0.9	-20.3	2.3	-58.2	1.9	-46.0
2.0	12.4	25.1	-2.7	64	-63.6	49	-49
2.1	9.6	20.7	-2.8	57.4	-7.4	45.2	-5.8
2.2	7.1	16.8	-3.3	49.9	-7.4	40.7	-6.6
2.3	4.7	13.2	-4.3	41.7	-10.2	35.3	-11.6
2.4	2.4	10.2	-5.7	33.3	-17.1	29.4	-17.2
2.5	0.0	7.7	-7.7	24.9	-24.9	23.3	-23.3

\* Values do not include dynamic load allowance.

**I. Develop  
Preliminary Tendon  
Profile**

Begin by determining the eccentricity of the tendon at primary locations and calculating the dead load and live load moments. Preliminary runs with assumed prestress losses are used to determine an appropriate tendon area or tendon force per foot. The prestress needs to provide zero tension stress in the slab at the service limit state. For this example, a 12 strand (0.6 inch diameter) tendon is found to be appropriate when spaced at 2'-5".

A handful of suppliers provide post-tensioning products in the U.S. Catalogs from the suppliers should be reviewed to ensure that standard tendons and ducts are used. LRFD Article 5.4.6.2 places maximum and minimum limits on the size of ducts based on the size of the tendon and the least concrete dimension of the member.

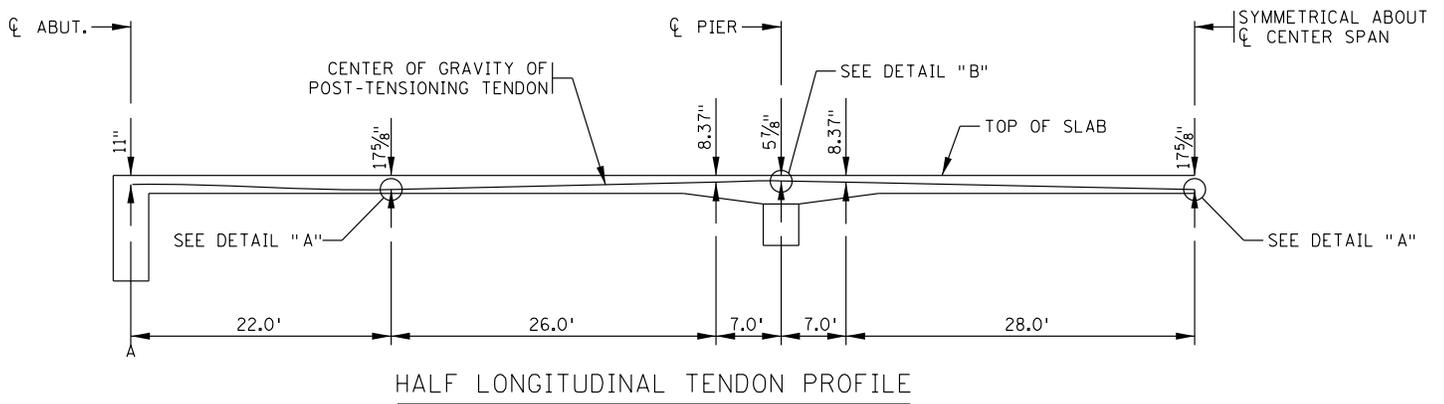
The tendon low points for the side spans will be placed at Span Points 1.4 and 3.6 (22 ft away from the abutment end of the span). The tendon low point for the center span will be placed at midspan (Span Point 2.5). The tendon high points will be located over the piers at Span Points 2.0 and 3.0. The tendon will be at the centroid of the gross cross section at each end of the structure (Span Points 1.0 and 4.0). See Figure 5.7.3.3 for a sketch of the proposed tendon profile and tendon centroid locations at high and low points of the tendon profile.

Critical points of the tendon geometry are calculated as:

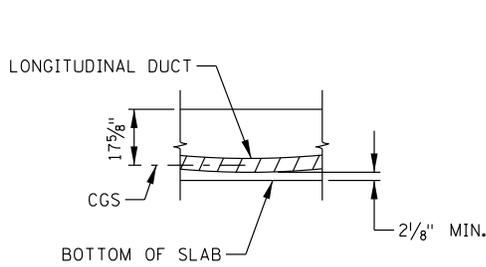
$$d_{\text{top}} \text{ at Span Point 1.0} = 11.00 \text{ in}$$

$$d_{\text{top}} \text{ at Span Points 1.4 and 2.5} = 22 - 1.5 - 0.625 - 2.25 = 17.63 \text{ in}$$

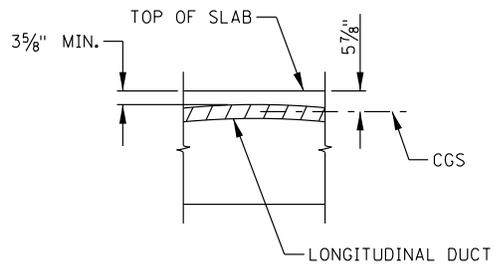
$$d_{\text{top}} \text{ at Span Points 2.0} = 3 + 0.625 + 2.25 = 5.88 \text{ in}$$



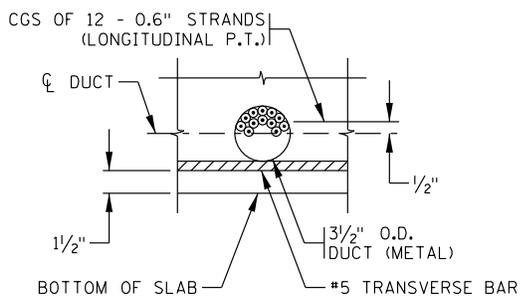
HALF LONGITUDINAL TENDON PROFILE



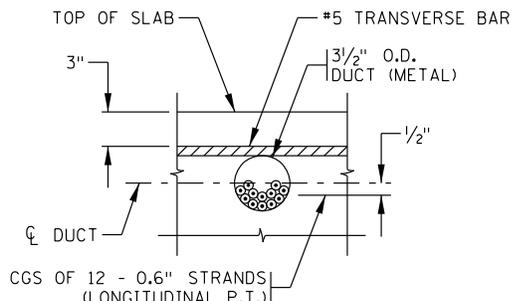
DETAIL "A"  
(AT LOW POINT)



DETAIL "B"  
(AT HIGH POINT)



AT LOW POINT OF P.T. DUCT PROFILE



AT HIGH POINT OF P.T. DUCT PROFILE

**Figure 5.7.3.3**  
**Tendon Profile and Centroid Locations**

### Tendon Equations

The tendon profile can be defined with a series of parabolas where for each parabola:

$$y = a \cdot x^2 + b \cdot x + c$$

With the section depth varying along the slab, use the top of the slab as the datum for defining the parabolic curves. The tendon profile is described with three parabolas; one describing the positive moment region of the side spans, a second describing the negative moment regions over the piers, and lastly a third parabola describing the positive moment region of the center span. Using the constraints:

$$\begin{aligned} y &= 11.00 \text{ inches at } x = 0 \text{ feet} \\ y &= 17.63 \text{ inches at } x = 22 \text{ feet} \\ \text{Slope} &= 0 \text{ at } x = 22 \text{ feet} \end{aligned}$$

The equation for the parabola for the positive moment regions of the side spans is found to be:

$$y = (-0.01369) \cdot x^2 + (0.6023) \cdot x + 11.00 \quad (x \text{ in feet, } y \text{ in inches})$$

Knowing that the y-coordinate and the slope for the tendon profile needs to be consistent at the location where parabolas meet, the second and third parabolas can be found.

Set the origin for the second parabola to be at Span Point 2.0. The following constraints can be used to determine the constants for the parabola:

$$\begin{aligned} y &= 5.875 \text{ inches at } x = 0 \text{ feet} \\ \text{Slope} &= 0 \text{ at } x = 0 \text{ feet} \\ y &\text{ at the end of the curve matches that of the 1}^{\text{st}} \text{ parabola} \\ \text{Slope at the end of the curve} &\text{ matches that of the 1}^{\text{st}} \text{ parabola} \end{aligned}$$

The location where the 1<sup>st</sup> and 2<sup>nd</sup> parabolas meet was found by changing the length of the 2<sup>nd</sup> parabola until the y value and slope matched that of the 1<sup>st</sup> parabola. The parabolas satisfy the criteria if they meet at a point 7.00 feet away from the pier (Span Point 1.873). The equation for the 2nd parabola is:

$$y = (0.05092) \cdot x^2 + (0) \cdot x + 5.875 \quad (x \text{ in feet, } y \text{ in inches})$$

With the 2<sup>nd</sup> parabola defined, the same procedure can be used to determine the constants for the 3<sup>rd</sup> parabola. With  $x = 0$  at Span Point 2.5, the constants are:

$$y = 17.625 \text{ inches at } x = 0 \text{ feet}$$

$$\text{Slope} = 0 \text{ at } x = 0 \text{ feet}$$

$$y \text{ at the end of the curve matches that of the 2}^{\text{nd}} \text{ parabola}$$

$$\text{Slope at the end of the curve matches that of the 2}^{\text{nd}} \text{ parabola}$$

After iterating the length of the 3<sup>rd</sup> parabola, the location where the  $y$  values and slopes match for the 2<sup>nd</sup> and 3<sup>rd</sup> parabolas is at a location 7.00 feet away from the pier (Span Point 2.1). The equation for the 2<sup>nd</sup> parabola is:

$$y = (-0.0118) \cdot x^2 + (0) \cdot x + 17.625 \quad (x \text{ in feet, } y \text{ in inches})$$

### **Tendon Geometry**

The tendon profile information for different points along the bridge are presented in Table 5.7.3.5. The equations presented above are in mixed units with the  $y$  values in inches and the  $x$  values in feet. To arrive at the tendon slopes in radians, the equation constants were divided by 12.

**Table 5.7.3.5**  
**Tendon Geometry**

Span Point	Depth of Section (in)	Section Centroid (in)	* Tendon Centroid (in)	Tendon Eccentricity (in)	Tendon Slope (radians)	Cumulative Length of Tendon (ft)	Cumulative Angle Change (radians)
1.0/4.0	22.00	11.00	11.00	0.00	+/- 0.050	0.000/180.094	0.000/0.559
1.1/3.9	22.00	11.00	13.90	-2.90	+/- 0.038	5.505/174.588	0.013/0.546
1.2/3.8	22.00	11.00	15.97	-4.97	+/- 0.025	11.008/169.086	0.025/0.534
1.3/3.7	22.00	11.00	17.21	-6.21	+/- 0.013	16.509/163.585	0.038/0.521
1.4/3.6	22.00	11.00	17.63	-6.63	0.000	22.009/158.084	0.050/0.508
1.5/3.5	22.00	11.00	17.21	-6.21	-/+ 0.013	27.509/152.584	0.063/0.496
1.6/3.4	22.00	11.00	15.97	-4.97	-/+ 0.025	33.010/147.083	0.075/0.483
1.7/3.3	22.00	11.00	13.90	-2.90	-/+ 0.038	38.513/141.581	0.088/0.471
1.8/3.2	22.96	11.48	11.00	0.48	-/+ 0.050	44.018/136.075	0.100/0.458
1.873/3.127	25.52	12.76	8.37	4.39	-/+ 0.059	48.024/132.069	0.110/0.449
1.9/3.1	26.48	13.24	7.42	5.82	-/+ 0.047	49.527/130.567	0.122/0.436
2.0/3.0	30.00	15.00	5.88	9.13	0.000	55.029/125.065	0.169/0.390
2.1/2.9	25.52	12.76	8.37	4.39	+/- 0.059	62.033/118.061	0.224/0.334
2.2/2.8	22.00	11.00	12.42	-1.42	+/- 0.041	69.041/111.053	0.238/0.321
2.3/2.7	22.00	11.00	15.31	-4.31	+/- 0.028	76.045/104.049	0.252/0.307
2.4/2.6	22.00	11.00	17.05	-6.05	+/- 0.014	83.047/97.047	0.266/0.293
2.5	22.00	11.00	17.63	-6.63	0.000	90.047	0.279

\* Measured from top of structural slab.

**J. Initial Prestress Losses**

Calculate the prestress losses due to friction, anchor set, and elastic shortening.

**[5.9.5.2.2]****Friction Losses**

An exponential equation is used to determine the friction losses at different tendon locations.

$$\Delta f_{pF} = f_{pj} \cdot \left[ 1 - e^{-(K \cdot x + \mu \cdot \alpha)} \right]$$

where:

$f_{pj}$  = stress in prestressing steel at jacking (ksi)

$x$  = length of prestressing tendon between any two points (ft)

$K$  = wobble coefficient from LRFD Table 5.9.5.2.2b-1 Use 0.0002

$\mu$  = coefficient of friction from LRFD Table 5.9.5.2.2b-1 Use 0.25

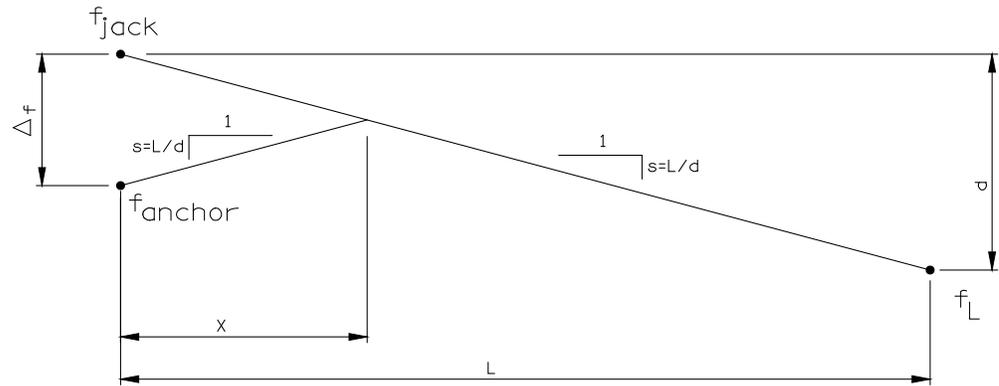
$\alpha$  = absolute value of angular change of prestressing path between two points (radians)

The friction coefficients assume that the strands are installed in rigid galvanized ducts.

The ratio of the force in the tendon to the force at any location after friction losses (Friction Factor) is summarized in Table 5.7.3.6.

**[5.9.5.2.1]****Anchor Set Losses**

The release of the tensioning jack from the PT tendon is accomplished by engaging strand wedges in the permanent anchor plate. A small shortening displacement in the tendon is necessary to seat the wedges. During construction, the tendon displacement is dependent on the jacking equipment used (some jacks can power seat wedges, others cannot). For design, a typical seating displacement is assumed (a standard value is 0.375 inches). The effective tension in the post-tensioning tendons at the jacking end is reduced due to the shortening of the tendon. This localized loss in tendon stress is called anchor set. The effect of anchor set is represented in Figure 5.7.3.4.



**Figure 5.7.3.4**

Assume the anchor set is 0.375 inches and use the friction losses at the dead end of the tendon to compute “d”. Assume the tendons are stressed to 80% of GUTS (Guaranteed Ultimate Tensile Strength):

$$f_{jack} = 0.80 \cdot f_{pu} = 0.80 \cdot 270.0 = 216.0 \text{ ksi}$$

The jacking stress at the dead end after friction losses is:

$$f_L = f_{jack} \cdot (\text{friction factor}) = 216.0 \cdot 0.839 = 181.2 \text{ ksi}$$

“d” represents the friction loss between the two end points:

$$d = f_{jack} - f_L = 216.0 - 181.2 = 34.8 \text{ ksi}$$

With “L” and “d” determined, the slope of the friction loss line is known. The increase in stress in the tendon as one moves away from the jacking end is assumed to have the same slope. With that assumption, the relationship between stress loss at the anchor and the location where the anchor loss dissipates can be found:

$$x = \left( \frac{\Delta f}{2} \right) \cdot \left( \frac{L}{d} \right)$$

The change in stress due to anchor set is found with Hooke’s law:

$$\sigma = \frac{\Delta f}{2} = E \cdot \epsilon = E \cdot \left( \frac{\Delta L}{12 \cdot X} \right)$$

Which can be substituted into the earlier equation:

$$X = \left( E \cdot \frac{\Delta_L}{12 \cdot X} \right) \cdot \left( \frac{L}{d} \right)$$

Which leads to:

$$X^2 = \frac{E \cdot \Delta_L \cdot L}{12 \cdot d} = \frac{28,500 \cdot 0.375 \cdot 180.094}{12 \cdot 34.8} = 4609$$

and

$$X = 67.9 \text{ ft}$$

Which, when put into the Hooke's law, determines the change in stress due to anchor set:

$$\Delta_f = 2 \cdot \left( \frac{E \cdot \Delta_L}{12 \cdot X} \right) = \left( \frac{E \cdot \Delta_L}{6 \cdot X} \right) = \frac{28,500 \cdot 0.375}{6 \cdot 67.9} = 26.2 \text{ ksi}$$

The stress at the anchor is:

$$f_{\text{anchor}} = f_{\text{jack}} - \Delta_f = 216.0 - 26.2 = 189.8 \text{ ksi}$$

The stress in the tendon between the anchor and point "X" can be found with interpolation.

### [5.9.5.2.3]

#### Elastic Shortening Losses

Elastic shortening losses for post-tensioned structures vary with the number of tendons used and the jacking processes followed. The LRFD Specifications provide a straightforward equation to estimate the prestress losses associated with elastic shortening for design.

$$\Delta f_{\text{pES}} = 0.25 \cdot \frac{E_p}{E_{ci}} \cdot f_{\text{cgp}}$$

The concrete stress at the height of the tendon when the slab is subjected to only dead load and prestress forces is computed for Span Point 2.5 in Section N and found to be:

$$\begin{aligned} f_{\text{cgp}} &= \frac{P_T}{A} + \frac{P_T \cdot e^2}{I} - \frac{M_{\text{DL}} \cdot e}{I} \\ &= \frac{211.1}{264} + \frac{211.1 \cdot 6.63^2}{10,650} - \frac{(52 + 19.8) \cdot 12 \cdot 6.63}{10,650} = 1.1 \text{ ksi} \end{aligned}$$

$$\Delta f_{\text{pES}} = 0.25 \cdot \frac{28,500}{3865} \cdot 1.1 = 2.0 \text{ ksi}$$

Table 5.7.3.6 summarizes the stresses in the tendon at tenth point span point locations. Losses associated with friction, anchor set, elastic shortening, shrinkage, creep, and relaxation (see Part N of this example for calculation of Shrinkage, Creep, and Relaxation losses) are presented. Initial and final tendon stresses are also presented.

**Table 5.7.3.6**  
**Tendon Stresses (ksi)**

Span Point	Friction Factor	Jacking Stress	Anchor Set Loss	Net Stress	Reversed Net Stress	Average Tendon Stress	Elastic Shortening Loss	Initial Tendon Stress	Shrinkage Loss	Creep Loss	Relaxation Loss	Final Tendon Stress
1.0	1.000	216.0	26.2	189.8	181.2	185.5	2.0	183.5	4.5	13.0	4.7	161.3
1.1	0.996	215.1	24.1	191.0	182.0	186.5	2.0	184.5	4.5	13.0	4.7	162.3
1.2	0.992	214.2	22.0	192.2	182.7	187.5	2.0	185.5	4.5	13.0	4.7	163.3
1.3	0.987	213.3	19.8	193.4	183.5	188.5	2.0	186.5	4.5	13.0	4.7	164.3
1.4	0.983	212.4	17.7	194.7	184.3	189.5	2.0	187.5	4.5	13.0	4.7	165.3
1.5	0.979	211.5	15.6	195.9	185.1	190.5	2.0	188.5	4.5	13.0	4.7	166.3
1.6	0.975	210.6	13.5	197.1	185.9	191.5	2.0	189.5	4.5	13.0	4.7	167.3
1.7	0.971	209.7	11.3	198.3	186.7	192.5	2.0	190.5	4.5	13.0	4.7	168.3
1.8	0.967	208.8	9.2	199.6	187.5	193.5	2.0	191.5	4.5	13.0	4.7	169.3
1.9	0.960	207.4	7.1	200.3	188.7	194.5	2.0	192.5	4.5	13.0	4.7	170.3
2.0	0.948	204.8	5.0	199.8	191.1	195.5	2.0	193.5	4.5	13.0	4.7	171.3
2.1	0.934	201.7	2.3	199.4	194.0	196.7	2.0	194.7	4.5	13.0	4.7	172.5
2.2	0.929	200.7	0.0	200.7	195.0	197.9	2.0	195.9	4.5	13.0	4.7	173.7
2.3	0.925	199.8	0.0	199.8	195.9	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.4	0.920	198.8	0.0	198.8	196.9	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.5	0.916	197.8	0.0	197.8	197.8	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.6	0.911	196.9	0.0	196.9	198.8	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.7	0.907	195.9	0.0	195.9	199.8	197.8	2.0	195.8	4.5	13.0	4.7	173.6
2.8	0.903	195.0	0.0	195.0	200.7	197.9	2.0	195.9	4.5	13.0	4.7	173.7
2.9	0.898	194.0	0.0	194.0	199.4	196.7	2.0	194.7	4.5	13.0	4.7	172.5
3.0	0.885	191.1	0.0	191.1	199.8	195.5	2.0	193.5	4.5	13.0	4.7	171.3
3.1	0.874	188.7	0.0	188.7	200.3	194.5	2.0	192.5	4.5	13.0	4.7	170.3
3.2	0.868	187.4	0.0	187.4	199.6	193.5	2.0	191.5	4.5	13.0	4.7	169.3
3.3	0.864	186.7	0.0	186.7	198.3	192.5	2.0	190.5	4.5	13.0	4.7	168.3
3.4	0.860	185.9	0.0	185.9	197.1	191.5	2.0	189.5	4.5	13.0	4.7	167.3
3.5	0.857	185.1	0.0	185.1	195.9	190.5	2.0	188.5	4.5	13.0	4.7	166.3
3.6	0.853	184.3	0.0	184.3	194.7	189.5	2.0	187.5	4.5	13.0	4.7	165.3
3.7	0.850	183.5	0.0	183.5	193.4	188.5	2.0	186.5	4.5	13.0	4.7	164.3
3.8	0.846	182.7	0.0	182.7	192.2	187.5	2.0	185.5	4.5	13.0	4.7	163.3
3.9	0.842	182.0	0.0	182.0	191.0	186.5	2.0	184.5	4.5	13.0	4.7	162.3
4.0	0.839	181.2	0.0	181.2	189.8	185.5	2.0	183.5	4.5	13.0	4.7	161.3

**K. Check Stress  
Limits on  
Prestressing  
Strands  
[Table 5.9.3-1]**

Stress Limits for Prestressing Strands:		
Prior to seating		$f_s \leq 0.90 \cdot f_{py} = 218.7 \text{ ksi}$
At anchorages after anchor set		$f_s \leq 0.70 \cdot f_{pu} = 189.0 \text{ ksi}$
End of seating zone after anchor set		$f_s \leq 0.74 \cdot f_{pu} = 199.8 \text{ ksi}$
At service limit after losses		$f_s \leq 0.80 \cdot f_{py} = 194.4 \text{ ksi}$

A review of the values in Table 5.7.3.6 indicates that none of the stress limits are exceeded.

**L. Summary of  
Analysis Results**

From this point forward, the design of an interior strip at points of maximum positive and negative moment subject to dead and live loads will be presented. The design procedure for the edge strip is similar. A summary of bending moments obtained at different locations along the superstructure for a 1 foot wide design strip is presented in Table 5.7.3.7. The analysis results are symmetric about midspan of the center span.

**Secondary Post-Tensioning Forces**

The linear haunches complicate the analysis of the slab superstructure. The nonprismatic section, combined with the parabolic tendon profiles prevent an easy hand method from being used to determine the secondary moments associated with post-tensioning a continuous superstructure. Therefore, a beam analysis program was used to determine secondary post-tensioning forces.

The tensioning of the tendon redistributes the dead load reactions of the superstructure. For the design example the redistribution was an increase in the abutment reaction of 0.87 kips and a corresponding reduction in the pier reactions of 0.87 kips. This implies that the positive moment regions of the tendon profile introduced slightly more curvature into the superstructure than the negative moment regions. The secondary moments associated with the redistribution amount to a linearly increasing positive moment in the side spans (0.0 kip-ft at the abutments and 47.9 kip-ft at the piers). The secondary moment in the center span is a constant positive value of 47.9 kip-ft.

**Table 5.7.3.7**  
**Interior Strip Moment Summary (per foot)**

Span Point	MDC (kip-ft)	PT Secondary Moments (kip-ft)	* Truck + Lane (kip-ft)		* Tandem + Lane (kip-ft)		* 0.9 (Truck Tr + Lane) (kip-ft)
			Max.	Min.	Max.	Min.	
1.0	0	0.0	0	0	0	0	-
1.1	31	4.8	32.5	-6.3	28.9	-5.2	-
1.2	51	9.6	54.1	-12.5	49.2	-10.1	-
1.3	62	14.4	65.8	-18.6	61.3	-15.3	-
1.4	62	19.1	70.6	-24.8	66.1	-20.3	-
1.5	52	23.9	68.9	-31.1	64.6	-25.4	-
1.6	31	28.7	61.6	-37.4	57.6	-30.5	-
1.7	1	33.5	46.8	-43.5	45.4	-35.6	-35.4
1.8	-40	38.3	26.9	-49.9	29.4	-40.8	-45.4
1.9	-92	43.1	15.7	-59.7	13.8	-49.5	-62.3
2.0	-155	47.9	16.5	-72.8	13.8	-61.3	-86.4
2.1	-78	47.9	11.0	-48.3	15.1	-41.2	-56.9
2.2	-20	47.9	30.3	-37.0	32.5	-30.9	-35.0
2.3	22	47.9	52.2	-30.5	49.7	-25.6	-
2.4	47	47.9	66.3	-25.3	61.5	-21.5	-
2.5	55	47.9	70.0	-20.1	65.0	-17.4	-

\* Includes dynamic load allowance.

**M. Check Stress  
Limits on Concrete**

The service limit state stresses at each of the critical locations are evaluated using the general equation (compression +, tension -):

$$f = \frac{P}{A} + \frac{M_p}{S} + \frac{M_s}{S}$$

where  $M_p$  is the total prestress moment and  $M_s$  is the service moment. The stress limits are:

**[5.9.4]**

At Transfer

Tension	$f_t = 0$ ksi
Compression	$f_c \leq 0.60 f'_{ci} = 2.7$ ksi

At Final

Tension	$f_t = 0$ ksi
Compression	
DC+PT+LL+IM	$f_c \leq 0.60 f'_c = 3.0$ ksi
DC+PT	$f_c \leq 0.45 f'_c = 2.25$ ksi
$\frac{1}{2}$ (DC+PT)+LL+IM	$f_c \leq 0.40 f'_c = 2.0$ ksi

**Check Location 1.0 (Interior Strip)**

Unfactored DC and PT Secondary Moment = 0 kip-ft

Tendon stress at transfer = 183.5 ksi

Tendon stress at final =  $183.5 - 0.11 \cdot (0.80 \cdot 270) = 159.7$  ksi

(assumes 11% long term losses)

Area of strand per foot =  $\frac{12(0.217)}{2.42} = 1.078$  in<sup>2</sup>/ft

Prestress force at transfer:  $P_i = 183.5 \cdot 1.078 = 197.8$  kips

Prestress force at final:  $P_f = 159.7 \cdot 1.078 = 172.2$  kips

Prestress eccentricity:  $e = 0$  in

Concrete area:  $A = 12 \cdot 22 = 264$  in<sup>2</sup>

Concrete section modulus:  $S = \frac{12 \cdot 22^2}{6} = 968$  in<sup>3</sup>

Check the stress in the concrete. Because the dead and live load moment and the prestress eccentricity are all equal to zero, the top and bottom fiber concrete stress is the same:

At transfer:  $f_b = f_t = \frac{P_i}{A} = \frac{197.8}{264} = 0.75$  ksi < 2.7 ksi OK

At final:  $f_b = f_t = \frac{P_f}{A} = \frac{172.2}{264} = 0.65$  ksi < 2.0 ksi OK

**Check Location 1.4 (Interior Strip)**

Unfactored DC and PT Secondary Moment:

$$M_{DC+PT} = 62 + 19.1 = 81.1 \text{ kip-ft}$$

$$\text{Service I Moment: } M_{SI} = 62 + 19.1 + 70.6 = 151.7 \text{ kip-ft}$$

$$\text{Service III Moment: } M_{SIII} = 62 + 19.1 + 0.8 \cdot (70.6) = 137.6 \text{ kip-ft}$$

$$\text{Tendon stress at transfer} = 187.5 \text{ ksi}$$

$$\text{Tendon stress at final} = 187.5 - 0.11 \cdot (0.80 \cdot 270) = 163.7 \text{ ksi}$$

(assumes 11% long term losses)

$$\text{Area of strand per foot} = \frac{12(0.217)}{2.42} = 1.078 \text{ in}^2/\text{ft}$$

$$\text{Prestress force at transfer: } P_i = 187.5 \cdot 1.078 = 202.1 \text{ kips}$$

$$\text{Prestress force at final: } P_f = 163.7 \cdot 1.078 = 176.5 \text{ kips}$$

$$\text{Prestress eccentricity: } e = 6.63 \text{ in}$$

$$\text{Concrete area: } A = 12 \cdot 22 = 264 \text{ in}^2$$

$$\text{Concrete section modulus: } S = \frac{12 \cdot 22^2}{6} = 968 \text{ in}^3$$

Check the bottom fiber stress at transfer:

$$\begin{aligned} f_b &= \frac{P_i}{A} + \frac{P_i \cdot e}{S} - \frac{M_{DC+PT}}{S} = \frac{202.1}{264} + \frac{202.1 \cdot 6.63}{968} - \frac{81.1 \cdot 12}{968} \\ &= 1.14 \text{ ksi} < 2.7 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

Check the bottom fiber stress at final:

$$\begin{aligned} f_b &= \frac{P_f}{A} + \frac{P_f \cdot e}{S} - \frac{M_{SIII}}{S} = \frac{176.5}{264} + \frac{176.5 \cdot 6.63}{968} - \frac{137.6 \cdot 12}{968} \\ &= 0.17 \text{ ksi} > 0 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

Check the top fiber stress at transfer:

$$\begin{aligned} f_t &= \frac{P_i}{A} - \frac{P_i \cdot e}{S} + \frac{M_{DC+PT}}{S} = \frac{202.1}{264} - \frac{202.1 \cdot 6.63}{968} + \frac{81.1 \cdot 12}{968} \\ &= 0.39 \text{ ksi} > 0 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

Check the top fiber compressive stress at final:

For DC + PT + LL + IM,

$$\begin{aligned} f_t &= \frac{P_f}{A} + \frac{P_f \cdot e}{S} + \frac{M_{SI}}{S} = \frac{176.5}{264} - \frac{176.5 \cdot 6.63}{968} + \frac{151.7 \cdot 12}{968} \\ &= 1.34 \text{ ksi} < 3.0 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

By inspection, the compressive stresses due to DC + PT and  $\frac{1}{2}$  (DC + PT) + LL + IM are less than the allowables.

**Check Location 2.0 (Interior Strip)**

Unfactored DC and PT Secondary Moment:

$$M_{DC+PT} = -155 + 47.9 = -107.1 \text{ kip-ft}$$

$$\text{Service I Moment: } M_{SI} = -155 + 47.9 - 86.4 = -193.5 \text{ kip-ft}$$

$$\text{Service III Moment: } M_{SIII} = -155 + 47.9 + 0.8 \cdot (-86.4) = -176.2 \text{ kip-ft}$$

$$\text{Tendon stress at transfer} = 193.5 \text{ ksi}$$

$$\text{Tendon stress at final} = 193.5 - 0.11 \cdot (0.80 \cdot 270) = 169.7 \text{ ksi}$$

(assumes 11% long term losses)

$$\text{Area of strand per foot} = \frac{12(0.217)}{2.42} = 1.078 \text{ in}^2/\text{ft}$$

$$\text{Prestress force at transfer: } P_i = 193.5 \cdot 1.078 = 208.6 \text{ kips}$$

$$\text{Prestress force at final: } P_f = 169.7 \cdot 1.078 = 182.9 \text{ kips}$$

$$\text{Prestress eccentricity: } e = 9.13 \text{ in}$$

$$\text{Concrete area: } A = 12 \cdot 30 = 360 \text{ in}^2$$

$$\text{Concrete section modulus: } S = \frac{12 \cdot 30^2}{6} = 1800 \text{ in}^3$$

Check the top fiber stress at transfer:

$$\begin{aligned} f_t &= \frac{P_i}{A} + \frac{P_i \cdot e}{S} - \frac{M_{DC+PT}}{S} = \frac{208.6}{360} + \frac{208.6 \cdot 9.13}{1800} - \frac{107.1 \cdot 12}{1800} \\ &= 0.92 \text{ ksi} < 2.7 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

Check the top fiber stress at final:

$$\begin{aligned} f_t &= \frac{P_f}{A} + \frac{P_f \cdot e}{S} - \frac{M_{SIII}}{S} = \frac{182.9}{360} + \frac{182.9 \cdot 9.13}{1800} - \frac{176.2 \cdot 12}{1800} \\ &= 0.26 \text{ ksi} > 0 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

Check the bottom fiber stress at transfer:

$$\begin{aligned} f_b &= \frac{P_i}{A} - \frac{P_i \cdot e}{S} + \frac{M_{DC+PT}}{S} = \frac{208.6}{360} - \frac{208.6 \cdot 9.13}{1800} + \frac{107.1 \cdot 12}{1800} \\ &= 0.23 \text{ ksi} > 0 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

Check the bottom fiber compressive stress at final:

For DC + PT + LL + IM,

$$\begin{aligned} f_b &= \frac{P_f}{A} + \frac{P_f \cdot e}{S} + \frac{M_{SI}}{S} = \frac{182.9}{360} - \frac{182.9 \cdot 9.13}{1800} + \frac{193.5 \cdot 12}{1800} \\ &= 0.87 \text{ ksi} < 3.0 \text{ ksi} \quad \underline{\text{OK}} \end{aligned}$$

By inspection, the compressive stresses due to DC + PT and  $\frac{1}{2}$  (DC + PT) + LL + IM are less than the allowables.

**Check Location 2.5 (Interior Strip)**

Unfactored DC and PT Secondary Moment:

$$M_{DC+PT} = 55 + 47.9 = 102.9 \text{ kip-ft}$$

$$\text{Service I Moment: } M_{SI} = 55 + 47.9 + 70.0 = 172.9 \text{ kip-ft}$$

$$\text{Service III Moment: } M_{SIII} = 55 + 47.9 + 0.8 \cdot (70.0) = 158.9 \text{ kip-ft}$$

$$\text{Tendon stress at transfer} = 195.8 \text{ ksi}$$

$$\text{Tendon stress at final} = 195.8 - 0.11 \cdot (0.80 \cdot 270) = 172.0 \text{ ksi}$$

(assumes 11% long term losses)

$$\text{Area of strand per foot} = \frac{12(0.217)}{2.42} = 1.078 \text{ in}^2/\text{ft}$$

$$\text{Prestress force at transfer: } P_i = 195.8 \cdot 1.078 = 211.1 \text{ kips}$$

$$\text{Prestress force at final: } P_f = 172.0 \cdot 1.078 = 185.4 \text{ kips}$$

$$\text{Prestress eccentricity: } e = 6.63 \text{ in}$$

$$\text{Concrete area: } A = 12 \cdot 22 = 264 \text{ in}^2$$

$$\text{Concrete section modulus: } S = \frac{12 \cdot 22^2}{6} = 968 \text{ in}^3$$

Check the bottom fiber stress at transfer:

$$f_b = \frac{P_i}{A} + \frac{P_i \cdot e}{S} - \frac{M_{DC+PT}}{S} = \frac{211.1}{264} + \frac{211.1 \cdot 6.63}{968} - \frac{102.9 \cdot 12}{968}$$

$$= 0.97 \text{ ksi} < 2.7 \text{ ksi} \quad \underline{\text{OK}}$$

Check the bottom fiber stress at final:

$$f_b = \frac{P_f}{A} + \frac{P_f \cdot e}{S} - \frac{M_{SIII}}{S} = \frac{185.4}{264} + \frac{185.4 \cdot 6.63}{968} - \frac{158.9 \cdot 12}{968}$$

$$= 0.002 \text{ ksi} > 0 \text{ ksi} \quad \underline{\text{OK}}$$

Check the top fiber stress at transfer:

$$f_t = \frac{P_i}{A} - \frac{P_i \cdot e}{S} + \frac{M_{DC+PT}}{S} = \frac{211.1}{264} - \frac{211.1 \cdot 6.63}{968} + \frac{102.9 \cdot 12}{968}$$

$$= 0.63 \text{ ksi} > 0 \text{ ksi} \quad \underline{\text{OK}}$$

Check the top fiber compressive stress at final:

For DC + PT + LL + IM,

$$f_t = \frac{P_f}{A} + \frac{P_f \cdot e}{S} + \frac{M_{SI}}{S} = \frac{185.4}{264} - \frac{185.4 \cdot 6.63}{968} + \frac{172.9 \cdot 12}{968}$$

$$= 1.58 \text{ ksi} < 3.0 \text{ ksi} \quad \underline{\text{OK}}$$

By inspection, the compressive stresses due to DC + PT and  $\frac{1}{2}$  (DC + PT) + LL + IM are less than the allowables.

**N. Time-Dependent Losses (Refined Method) [5.9.5.4]**

Use Location 2.5 to calculate losses due to shrinkage, creep and relaxation because the highest effective prestressing force occurs at this location. This will result in conservative values for creep and relaxation losses.

**[5.9.5.4.2]**

**Shrinkage**

$$f_{pSR} = (13.5 - 0.125 \cdot H)$$

H = relative humidity (use 73%)

$$f_{pSR} = [13.5 - 0.125 \cdot (73)] = 4.5 \text{ ksi}$$

**[5.9.5.4.3]**

**Creep**

The moment associated with the wear course and barriers for a 1 foot wide section of slab is 4 kip-ft.

$$\Delta f_{pCR} = 12.0 \cdot f_{cgp} - 7.0 \cdot \Delta f_{cdp}$$

$$f_{cgp} = 1.1 \text{ ksi}$$

(calculated earlier in Part J)

$$\Delta f_{cdp} = \frac{M_{DW} \cdot e}{I} = \frac{4 \cdot (12) \cdot 6.63}{10,650} = 0.030 \text{ ksi}$$

$$\Delta f_{pCR} = 12.0 \cdot (1.1) - 7.0 \cdot (0.030) = 13.0 \text{ ksi}$$

**[5.9.5.4.4]**

**Relaxation**

For low-relaxation strands:

$$0.30 \cdot \Delta f_{pR2} = 0.30 \cdot [20.0 - 0.3 \cdot \Delta f_{pF} - 0.4 \cdot \Delta f_{pES} - 0.2 \cdot (\Delta f_{pSR} + \Delta f_{pCR})]$$

If the friction losses are such that the tendon stresses after jacking are above  $0.70 \cdot f_{pu}$ , then  $\Delta f_{pF}$  is assumed equal to zero.

$$= 0.30 \cdot [20.0 - 0.3 \cdot (0) - 0.4 \cdot (2.0) - 0.2 \cdot (4.5 + 13.0)]$$

$$= 4.7 \text{ ksi}$$

**Total Losses**

Total time-dependent losses =  $4.5 + 13.0 + 4.7 = 22.2 \text{ ksi}$   
(10.3% of  $0.80 \cdot f_{pu}$ )

The computed losses of 10.3% are approximately the same as those assumed in the stress checks (11%). If the computed losses are significantly different from the assumed, designers will need to recalculate the stresses based on a new assumed loss and iterate until the computed and assumed losses converge.

**O. Investigate  
Strength Limit  
State – Flexure  
[5.7.3.3.2]**

The flexural strength of the slab needs to be sufficient to carry factored loads associated with the strength limit state and also satisfy the maximum and minimum reinforcement checks.

**Check Location 1.4 (Interior Strip)**

Compute the Strength I design moment for a 1 foot wide strip of slab:

$$M_U = 1.0 \cdot [1.25 \cdot (62) + 1.00 \cdot (19.1) + 1.75 \cdot (70.6)] = 220 \text{ kip-ft}$$

Determine the theoretical cracking moment for the cross section ( $M_{cr}$ ).

To compute the maximum cracking moment, use the prestress force at transfer (202.1 kips).

Solve for the moment that produces  $f_r$  at the bottom of the section:

$$M_{cr} = (f_r + f_{PTS}) \cdot S$$

**[5.4.2.6]**

The assumed rupture or cracking stress for concrete is:

$$f_r = 0.24 \cdot \sqrt{f'_c} = 0.24 \cdot \sqrt{5.0} = 0.537 \text{ ksi}$$

The stress due to prestressing (including secondary moments) is:

$$\begin{aligned} f_{PTS} &= \frac{P}{A} + \frac{P \cdot e}{S} - \frac{M_{\text{secondary}}}{S} \\ &= \frac{202.1}{264} + \frac{202.1 \cdot 6.63}{968} - \frac{19.1 \cdot 12}{968} = 1.913 \text{ ksi} \end{aligned}$$

The cracking moment is:

$$M_{cr} = 2372 \text{ kip-in} = 198 \text{ kip-ft}$$

$$1.2M_{cr} = 238 \text{ kip-ft}$$

**[5.7.3.2]  
[5.7.3.1.1]**

Compute the capacity neglecting any benefit from mild steel. Use the equations for bonded tendons:

$$k = 0.28 \text{ (LRFD Table C5.7.3.1.1-1)}$$

$$d_p = 11.00 + 6.625 = 17.625 \text{ in}$$

$$c = \left( \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \right)$$

$$= \left( \frac{10.78 \cdot 270}{0.85 \cdot 5.0 \cdot 0.80 \cdot 12 + 0.28 \cdot 1.078 \cdot \frac{270}{17.625}} \right) = 6.41 \text{ in}$$

$$f_{ps} = f_{pu} \cdot \left( 1 - k \cdot \frac{c}{d_p} \right) = 270 \cdot \left( 1 - 0.28 \cdot \frac{6.41}{17.625} \right) = 242.5 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.80 \cdot 6.41 = 5.13 \text{ in}$$

The flexural resistance can be computed as:

$$\begin{aligned} \phi \cdot M_n &= \phi \cdot A_{ps} \cdot f_{ps} \cdot \left( d_p - \frac{a}{2} \right) = 1.0 \cdot 1.078 \cdot 242.5 \cdot \left( 17.625 - \frac{5.13}{2} \right) \\ &= 3937 \text{ kip-in} = 328 \text{ kip-ft} \end{aligned}$$

which is greater than  $1.2 \cdot M_{cr}$  (238 kip-ft) and  $M_u$  (220 kip-ft)

### Check Location 2.0 (Interior Strip)

Using the moments given in Table 5.7.3.7:

$$M_u = 1.0 \cdot [1.25 \cdot (155) + 1.00 \cdot (47.9) + 1.75 \cdot (86.4)] = 393 \text{ kip-ft}$$

Solve for the moment that produces  $f_r$  at the bottom of the section:

$$\begin{aligned} f_b = f_r &= \frac{P}{A} + \frac{P \cdot e}{S} + \frac{M_{\text{secondary}}}{S} - \frac{M_{cr}}{S} \\ -0.537 &= \frac{208.6}{360} + \frac{208.6 \cdot 9.13}{1800} + \frac{47.9 \cdot 12}{1800} - \frac{M_{cr}}{1800} \end{aligned}$$

$$M_{cr} = 4489 \text{ kip-in} = 374 \text{ kip-ft}$$

$$1.2 \cdot M_{cr} = 449 \text{ kip-ft}$$

### [5.7.3.2]

Compute the capacity neglecting any benefit from mild steel.

$$k = 0.28 \text{ (LRFD Table C5.7.3.1-1)}$$

$$d_p = 15.00 + 9.13 = 24.13 \text{ in}$$

$$\begin{aligned} c &= \left( \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \right) \\ &= \left( \frac{1.078 \cdot 270}{0.85 \cdot 5.0 \cdot 0.80 \cdot 12 + 0.28 \cdot 1.078 \cdot \frac{270}{24.13}} \right) = 6.59 \text{ in} \end{aligned}$$

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 270 \cdot \left(1 - 0.28 \cdot \frac{6.59}{24.13}\right) = 249.4 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.80 \cdot 6.59 = 5.27 \text{ in}$$

The flexural resistance can be computed as:

$$\begin{aligned} \phi \cdot M_n &= \phi \cdot A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) = 1.0 \cdot 1.078 \cdot 249.4 \cdot \left(24.13 - \frac{5.27}{2}\right) \\ &= 5779 \text{ kip-in} = 482 \text{ kip-ft} \end{aligned}$$

which is greater than  $1.2 \cdot M_{Cr}$  (449 kip-ft) and  $M_U$  (393 kip-ft).

### Check Location 2.5 (Interior Strip)

Using the moments given in Table 5.7.3.7:

$$M_U = 1.0 \cdot [1.25 \cdot (55) + 1.00 \cdot (47.9) + 1.75 \cdot (70)] = 239 \text{ kip-ft}$$

Solve for the moment that produces  $f_r$  at the bottom of the section:

$$\begin{aligned} f_b = f_r &= \frac{P}{A} + \frac{P \cdot e}{S} + \frac{M_{\text{secondary}}}{S} - \frac{M_{Cr}}{S} \\ -0.537 &= \frac{211.1}{264} + \frac{211.1 \cdot 6.63}{968} + \frac{47.9 \cdot 12}{968} - \frac{M_{Cr}}{968} \\ M_{Cr} &= 2119 \text{ kip-in} = 177 \text{ kip-ft} \\ 1.2 \cdot M_{Cr} &= 212 \text{ kip-ft} \end{aligned}$$

### [5.7.3.2]

Compute the capacity neglecting any benefit from mild steel.

$$k = 0.28 \text{ (LRFD Table C5.7.3.1-1)}$$

$$d_p = 11.00 + 6.625 = 17.625 \text{ in}$$

$$\begin{aligned} c &= \left( \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_c \cdot \beta_1 \cdot b + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \right) \\ &= \left( \frac{1.078 \cdot 270}{0.85 \cdot 5.0 \cdot 0.80 \cdot 12 + 0.28 \cdot 1.078 \cdot \frac{270}{17.625}} \right) = 6.41 \text{ in} \end{aligned}$$

$$f_{ps} = f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 270 \cdot \left(1 - 0.28 \cdot \frac{6.41}{17.625}\right) = 242.5 \text{ ksi}$$

$$a = \beta_1 \cdot c = 0.80 \cdot 6.41 = 5.13 \text{ in}$$

The flexural resistance can be computed as:

$$\begin{aligned}\phi \cdot M_n &= \phi \cdot A_{ps} \cdot f_{ps} \cdot \left( d_p - \frac{a}{2} \right) = 1.0 \cdot 1.078 \cdot 242.5 \cdot \left( 17.625 - \frac{5.13}{2} \right) \\ &= 3937 \text{ kip-in} = 328 \text{ kip-ft}\end{aligned}$$

which is greater than  $M_u$  (239 kip-ft) and  $1.2 \cdot M_{cr}$  (212 kip-ft).

### Check Principal Stresses

[Future manual content]

#### [5.7.3.3.1]

### Check Maximum Reinforcement

The maximum amount of reinforcement permitted in a section is based on the ratio of the depth of the section in compression compared to the depth of the distance to the tension reinforcement from the compression side of the section. The ratio can be no more than 0.42. When calculating "d", do not include the wearing course.

For Span Point 1.4:

$$\frac{c}{d} = \frac{6.41}{17.625} = 0.36 \quad \underline{\text{OK}}$$

For Span Point 2.0:

$$\frac{c}{d} = \frac{6.59}{24.13} = 0.27 \quad \underline{\text{OK}}$$

For Span Point 2.5:

$$\frac{c}{d} = \frac{6.41}{17.625} = 0.36 \quad \underline{\text{OK}}$$

**P. Shear**  
**[5.13.3.6]**

The shear force components for a typical 1 foot wide strip of slab are summarized in Table 5.7.3.8.

**Table 5.7.3.8**  
**Shear Summary (per foot)**

Span Point	$V_{DC}$ (kips)	PT Secondary Shear (kips)	* Truck + Lane (kips)		* Tandem + Lane (kips)	
			Max	Min	Max	Min
1.0	6.5	0.87	4.9	-0.8	4.3	-0.7
1.1	4.7	0.87	4.1	-0.8	3.6	-0.7
1.2	2.8	0.87	3.4	-0.8	3.0	-1.0
1.3	0.9	0.87	2.7	-1.3	2.5	-1.5
1.4	-0.9	0.87	2.0	-2.0	1.9	-2.0
1.5	-2.8	0.87	1.5	-2.6	1.5	-2.5
1.6	-4.6	0.87	1.0	-3.3	1.1	-3.0
1.7	-6.5	0.87	0.6	-4.0	0.7	-3.5
1.8	-8.3	0.87	0.2	-4.7	0.4	-4.0
1.9	-10.4	0.87	0.2	-5.3	0.2	-4.4
2.0	12.4	0.0	6.0	-4.7	4.9	-3.7
2.1	9.6	0.0	5.2	-0.7	4.4	-0.6
2.2	7.1	0.0	4.5	-0.7	3.8	-0.7
2.3	4.7	0.0	3.7	-1.0	3.3	-1.1
2.4	2.4	0.0	2.9	-1.5	2.7	-1.5
2.5	0.0	0.0	2.2	-2.2	2.1	-2.1

\* Includes dynamic load allowance.

The LRFD Specifications do not require that a shear check be performed, however MnDOT design practice is to do so.

To minimize the effort associated with the shear check, conservatively check the largest design shear force on a non-haunch portion of the slab. If the check is satisfied, all sections of the slab can be considered adequate. If the check is not satisfied additional investigation is necessary.

The Strength I design shear at Span Point 2.0 is:

$$V_u = 1.25 \cdot (12.4) + 1.00 \cdot (0.0) + 1.75 \cdot (6.0) = 26.0 \text{ kips}$$

Investigate LRFD Equation 5.8.3.4.2-2. No axial load is applied. Neglect any mild flexural reinforcement and any beneficial vertical prestressing effect. As a starting point, assume  $\theta$  is equal to 30 degrees.

$$\varepsilon_x = \left[ \frac{\frac{M_u}{d_v} + 0.5 \cdot N_u + 0.5 \cdot (V_u - V_p) \cdot \cot \theta - A_{ps} \cdot f_{po}}{E_s \cdot A_s + E_p \cdot A_{ps}} \right]$$

Use the  $M_u$  and  $d_v$  from Span Point 1.4

$$\begin{aligned} \varepsilon_x &= \frac{\frac{220 \cdot 12}{15.84} + 0.5 \cdot 0 + 0.5 \cdot 26 \cdot (\cot 30^\circ) - 1.078 \cdot 0.7 \cdot 270}{29,000 \cdot 0 + 28,500 \cdot 1.078} \\ &= -0.000244 \end{aligned}$$

Determine the crack spacing parameter next. Use  $d_v$  for  $s_x$

**[Eqn. 5.8.3.4.2-4]**

$$s_{xe} = s_x \cdot \frac{1.38}{a_g + 0.63} = 15.84 \cdot \frac{1.38}{1.5 + 0.63} = 10.3$$

With the strain and crack parameters determined, refer to Table 5.8.3.4.2-2 to determine the appropriate  $\beta$  and  $\theta$  values for use in computing the shear capacity of the concrete. Use the values in the cell for  $s_{xe} < 15$  and  $\varepsilon_x < -0.20$  ( $\beta = 5.34$  and  $\theta = 29.5^\circ$ ).

The required nominal shear capacity is:

$$V_n = \frac{V_u}{\phi_v} = \frac{26.0}{0.9} = 28.9 \text{ kips}$$

The shear capacity of the concrete is:

$$\begin{aligned} V_c &= 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 5.34 \cdot \sqrt{5.0} \cdot 12 \cdot 15.84 \\ &= 0.0316 \cdot 5.34 \cdot \sqrt{5.0} \cdot 12 \cdot 15.84 = 71.7 \gg 28.9 \text{ kips} \end{aligned}$$

**Q. Minimum  
Longitudinal  
Reinforcement  
[5.8.3.5]**

[Future manual content]

**R. Distribution  
Reinforcement  
[5.14.4.1]**

The minimum amount of transverse reinforcement in a horizontal plane shall be taken as a percentage of the main reinforcement:

$$\frac{100}{\sqrt{L}} \cdot \frac{f_{pe}}{60} \leq 50\%$$

For Spans 1 and 3

$$\frac{100}{\sqrt{55}} \cdot \frac{170.3}{60} = 38\%$$

Interior Strip: Maximum positive moment

$$\text{Positive moment prestressing} = 1.078 \text{ in}^2/\text{ft}$$

$$\text{Transverse reinforcement} = 0.38 \cdot (1.078) = 0.41 \text{ in}^2/\text{ft}$$

Use #6 @ 12",  $A_s = 0.44 \text{ in}^2/\text{ft}$

**S. Shrinkage and  
Temperature  
Reinforcement  
[5.10.8.2]**

Using an average thickness of 26 inches, the required temperature steel is:

$$A_s \geq 0.11 \cdot \frac{A_g}{f_y} = 0.11 \cdot \left( \frac{12 \cdot 26}{60} \right) = 0.57 \text{ in}^2/\text{ft each direction, both faces}$$

Half should be placed in each face:

$$A_s = \frac{1}{2} \cdot (0.57) = 0.29 \text{ in}^2/\text{ft each direction}$$

Use #5 @ 12",  $A_s = 0.31 \text{ in}^2/\text{ft}$

**T. Deformations  
[2.5.2.6]**

**Dead Load Deflection Plus Prestress Camber**

The total weight of the superstructure is used for dead load deflections. The gross moment of inertias are used and a computer analysis is run to obtain instantaneous deflections. The results of the computer analysis, (dead load deflections and camber due to prestress) are presented below. Using the long-term multipliers (from Section 4.6 of the PCI Handbook with composite topping), the long-term deflections are calculated as:

	$\Delta$ at release (Spans 1 and 3) (in)	$\Delta$ at release (Span 2) (in)	Multiplier	$\Delta$ final (Spans 1 and 3) (in)	$\Delta$ final (Span 2) (in)
Prestress	+0.92	0.73	2.20	2.02	1.61
$W_{DC}$	-0.46	-0.50	2.40	-1.10	-1.20
Total	0.46	0.23		0.92	0.41

With a net upward deflection in all spans, the slab is cambered downward. The camber is equal to the camber at release plus  $\frac{1}{2}$  of the long-term camber.

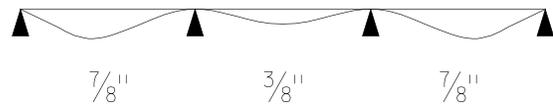
Spans 1 and 3:

$$0.46 + \frac{1}{2} \cdot (0.92) = 0.92 \text{ in} \qquad \text{Round down and use } \frac{7}{8} \text{ in}$$

Span 2:

$$0.23 + \frac{1}{2} \cdot (0.41) = 0.43 \text{ in} \qquad \text{Round down and use } \frac{3}{8} \text{ in}$$

Total Camber



**[2.5.2.6.2]**

**Live Load Deflections**

$$\begin{aligned} \text{Allowable } \Delta_{LL+I} &= \frac{\text{Span}}{800} \\ &= \frac{55 \cdot 12}{800} = 0.83 \text{ in (Span 1 and 3)} \\ &= \frac{70 \cdot 12}{800} = 1.05 \text{ in (Span 2)} \end{aligned}$$

**[3.6.1.3.2]**

Two live load cases are evaluated as part of the live load deflection check. One is the design truck alone. The other is lane load combined with 25% of the truck load deflection.

A computer analysis (based on gross nonprismatic section properties) had the following deflections for a full lane of live load:

Spans 1 and 3:

Truck deflection: 6.24 in/lane

Lane deflection: 2.59 in/lane

Truck check:

$$= (1 + IM) \cdot (\Delta_{\text{truck}}) \cdot (\text{distribution factor})$$

$$= 1.33 \cdot 6.24 \cdot 0.054 = 0.45 < 0.83 \text{ in}$$

OK

Lane/truck check:

$$= 0.25 \cdot (1 + IM) \cdot (\Delta_{\text{truck}} + \Delta_{\text{lane}}) \cdot (\text{distribution factor})$$

$$= 0.25 \cdot 1.33 \cdot (6.24 + 2.59) \cdot (0.054) = 0.25 < 0.83 \text{ in}$$

OK

Span 2:

Truck deflection: 8.83 in/lane

Lane deflection: 3.60 in/lane

Truck check:

$$= (1 + IM) \cdot (\Delta_{\text{truck}}) \cdot (\text{distribution factor})$$

$$= 1.33 \cdot 8.83 \cdot 0.054 = 0.63 < 1.05 \text{ in}$$

OK

Lane/truck check:

$$= 0.25 \cdot (1 + IM) \cdot (\Delta_{\text{truck}} + \Delta_{\text{lane}}) \cdot (\text{distribution factor})$$

$$= 0.25 \cdot 1.33 \cdot (8.83 + 3.60) \cdot (0.054) = 0.35 < 1.05 \text{ in}$$

OK

### **U. Anchorage Zone** **[5.10.9]**

Anchorage are designed at the strength limit state for the factored jacking force.

Due to the simplicity of the geometry of the anchorage and the lack of substantial deviation in the force flow path, the approximate procedure described in LRFD Article 5.10.9.6 is used.

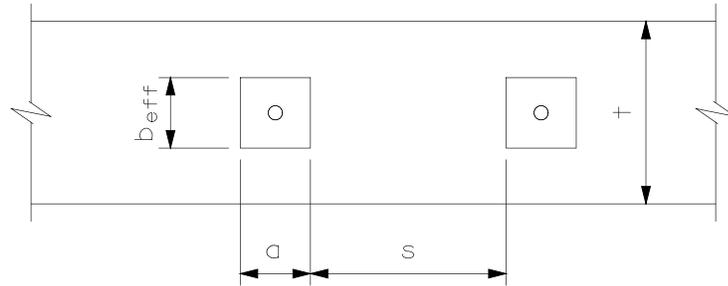
For a 12- 0.6" diameter strand tendon, use a square anchorage plate with a side dimension of 12.875 inches (a and b). Assume a duct outer diameter of 6.25 inches.

#### **[5.10.9.6.2]**

#### **General Zone Compressive Stresses**

Determine the allowable concrete compressive stress from:

$$f_{ca} = \frac{0.6 \cdot P_u \cdot K}{A_b \cdot \left[ 1 + \ell_c \cdot \left( \frac{1}{b_{\text{eff}}} - \frac{1}{t} \right) \right]}$$



**Figure 5.7.3.5**  
**Anchorage Dimensions**

Determine the value for  $K$  based on the spacing of the tendons and the size of the anchorage plate.

$$s = 29 > 2 \cdot a = 25.75 \quad \text{Use } K = 1$$

The jacking force is:

$$\begin{aligned} P_{\text{jack}} &= (\text{jacking stress}) \cdot (\# \text{ of strands}) \cdot A_{\text{strand}} \\ &= 216 \cdot 12 \cdot 0.217 = 562 \text{ kips} \end{aligned}$$

**[3.4.3.2]**

The factored tendon force for anchorage design is:

$$P_u = 1.2 \cdot (562) = 674.4 \text{ kips}$$

$$b_{\text{eff}} = 12.875 \text{ in}$$

$$l_c = 1.15 \cdot b_{\text{eff}} = 1.15 \cdot (12.875) = 14.81 \text{ in}$$

$$t = 22 \text{ in}$$

$$A_b = (a \cdot b_{\text{eff}}) - \frac{\pi \cdot D^2}{4} = (12.875)^2 - \frac{\pi \cdot 6.25^2}{4} = 135.1 \text{ in}^2$$

$$f_{ca} = \frac{0.6 \cdot 674.4 \cdot 1.0}{135.1 \cdot \left[ 1 + 14.81 \cdot \left( \frac{1}{12.875} - \frac{1}{22} \right) \right]} = 2.03 \text{ ksi}$$

**[5.10.9.3.1]**

The factored concrete compressive strength for the general zone shall not exceed  $0.7 \cdot \phi \cdot f'_{ci}$ .

$$\phi = 0.80 \text{ for compression in anchorage zones}$$

$$f_{ca} \leq 0.7 \cdot \phi \cdot f'_{ci} = 0.7 \cdot (0.8) \cdot 4.5 = 2.52 \text{ ksi}$$

Therefore, use  $f_{ca} = 2.03 \text{ ksi}$

Determine the compressive stress at a distance equal to the plate's smaller dimension. Assume the load distributes at an angle of  $30^\circ$ .

$$A_e = (a + 2 \cdot \tan 30^\circ \cdot a)^2 - \frac{\pi \cdot D^2}{4}$$

$$= (12.875 + 2 \cdot 0.577 \cdot 12.875)^2 - \frac{\pi \cdot 6.25^2}{4} = 739 \text{ in}^2$$

$$f_e = \frac{P_u}{A_e} = \frac{674.4}{739} = 0.90 \text{ ksi} < f_{ca} = 2.03 \text{ ksi} \quad \underline{\text{OK}}$$

**[5.10.9.6.3]**

**General Zone Bursting Force**

The tendon slope at the ends of the superstructure from Table 5.7.3.5 is 0.050 radians (3 degrees).

The bursting forces in the anchorage is calculated as:

$$T_{burst} = 0.25 \cdot P_u \cdot \left(1 - \frac{a}{h}\right) + 0.5 \cdot |P_u \cdot \sin \alpha|$$

$$= 0.25 \cdot (674.4) \cdot \left(1 - \frac{12.875}{22}\right) + 0.5 \cdot (674.4) \cdot 0.052 = 87.5 \text{ kips}$$

$$d_{burst} = 0.5 \cdot (h - 2 \cdot e) + 5 \cdot e \cdot (\sin \alpha) \quad (\text{for this example, } e = 0)$$

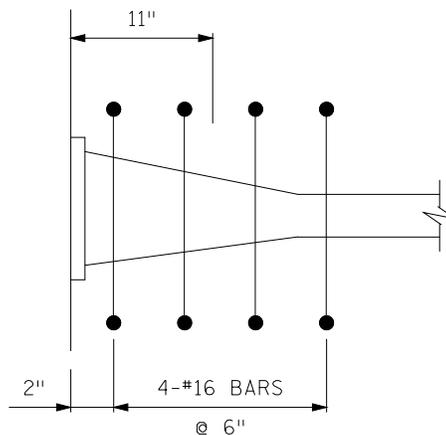
$$= 0.5 \cdot (22) = 11 \text{ in}$$

Using  $\phi = 1.00$  for tension in steel in anchorage zones, then  $\phi \cdot f_y = 1.0 \cdot (60) = 60 \text{ ksi}$ :

$$A_s \text{ req'd} = \frac{87.5}{60} = 1.46 \text{ in}^2 \text{ (spaced within } 2.5 \cdot d_{burst} = 27.5 \text{ in)}$$

Use 4 - #5 closed stirrups spaced at 6 inches (refer to Figure 5.7.3.6).

$$A_s = 4 \cdot (0.31) \cdot 2 \text{ legs/stirrup} = 2.48 > 1.46 \text{ in}^2 \quad \underline{\text{OK}}$$



**Figure 5.7.3.6**  
**Bursting Force Reinforcing**

**[5.10.9.3.2]****General Zone Edge Tension Forces**

Edge tension forces are the tensile forces in the anchorage zone acting close to the transverse edge (spalling forces) and longitudinal edges (longitudinal edge tension forces). For the case of a concentrically loaded anchorage zone, the longitudinal edge tension forces are insignificant, and the magnitude of the design spalling force may be taken as 2% of the total post-tensioning force.

$$\text{Spalling Force} = 0.02 \cdot (674.4) = 13.5 \text{ kips}$$

$$\text{Using } \phi \cdot f_y = 1.0 \cdot (60) = 60 \text{ ksi:}$$

$$A_s \text{ req'd} = \frac{13.5}{60} = 0.22 \text{ in}^2$$

Use 2- #5 bars,  $A_s = 0.62 \text{ in}^2$

**V. Summary of  
Final Design**

A summary of the primary reinforcement for the slab is provided in Figure 5.7.3.7. A typical transverse half section is illustrated for the midspan section and for the section over the piers.

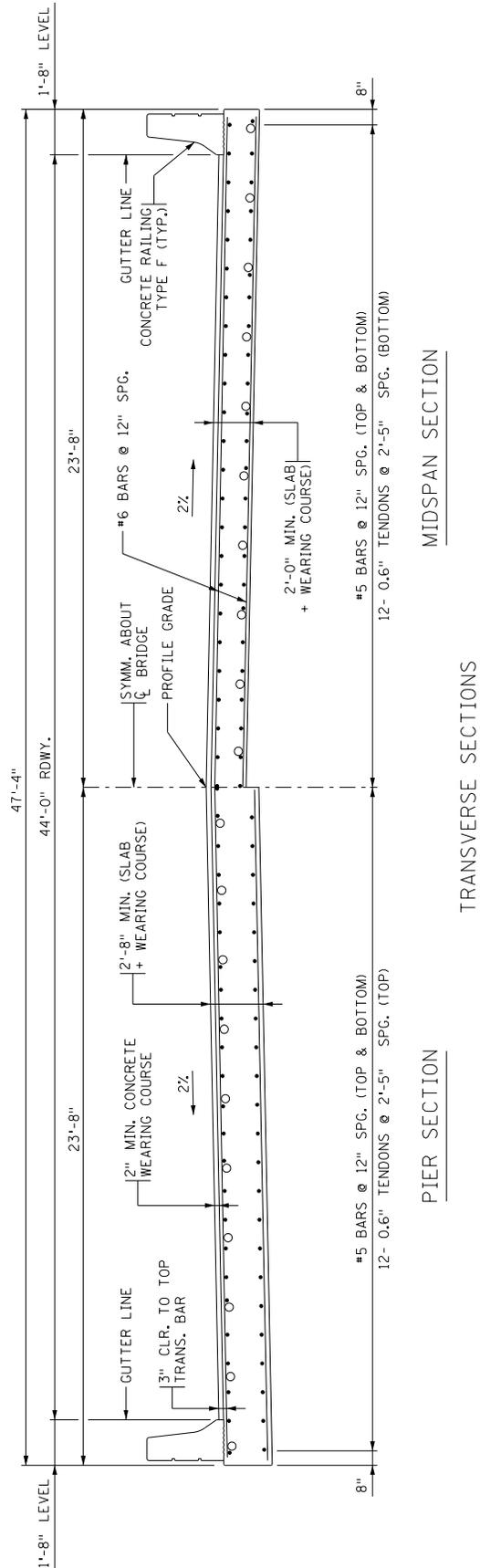


Figure 5.7.3.7

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APPENDIX 5-A

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

DECKS:

**Top Transverse Deck Bars**

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 for bar size and spacing. A Class A splice is provided where all top transverse bar splices occur between beams, with 50% of the bars spliced at a given location. A Class B splice is provided where 100% of the bars are spliced at a given location between beams or where 50% of the bars are spliced at a given location over beams. Avoid splicing 100% of bars over beams.

Top Transverse Deck Bar Lap Splice Lengths				
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	All Splices Between Beams and 50% are at Same Location ( <i>preferred</i> )	100% of Splices at Same Location Between Beams or 50% of Splices Over Beams at Same Location
3"	> 5"	#4	1'-6"	1'-11"
		#5	1'-10"	2'-5"
		#6	2'-2"	2'-10"
	5"	#4	1'-6"	1'-11"
		#5	1'-10"	2'-5"
		#6	2'-9"	3'-7"

**Top Longitudinal Deck Bars**

See LRFD Bridge Design Manual Table 9.2.1.1 & Figure 9.2.1.6 or Table 9.2.1.2 & Figure 9.2.1.7 for bar size and spacing. Detail reinforcement such that no more than 50% of top longitudinal bars are spliced at any cross-section through the deck (Class A splice).

Top Longitudinal Deck Bar Lap Splice Lengths		
Concrete Cover to Bar Being Considered	Bar Size	Lap Splice Length
$\geq 3 \frac{1}{2}$ "	#4	1'-6"
	#5	1'-10"
	#6	2'-2"

## APPENDIX 5-A (CONTINUED)

## MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

## DECKS: (cont'd)

**Bottom Transverse Deck Bars**

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 for bar size and spacing. A Class A splice is provided where all bottom transverse bars are spliced over beams, with 50% of the bars spliced at a given location. A Class B splice is provided where 100% of the bars are spliced at a given location over beams or where 50% of the bars are spliced at a given location between beams. Avoid splicing 100% of bars between beams.

Bottom Transverse Deck Bar Lap Splice Lengths				
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	All Splices Over Beams and 50% are at Same Location ( <i>preferred</i> )	100% of Splices at Same Location Over Beams or 50% of Splices Between Beams at Same Location
1"	≥ 4"	#4	1'-10"	2'-5"
		#5	2'-9"	3'-6"
		#6	3'-9"	4'-10"

**Bottom Longitudinal Deck Bars**

See LRFD Bridge Design Manual Table 9.2.1.1 or Table 9.2.1.2 & Figure 9.2.1.7 for bar size and spacing. A Class B splice is provided. Where possible, detail such that no more than 50% of the bottom longitudinal deck bars are spliced at a given cross-section through the deck.

Bottom Longitudinal Deck Bar Lap Splice Lengths			
Concrete Cover to Bar Being Considered	Bar Spacing	Bar Size	50% of Splices at Same Location ( <i>Preferred</i> ) or 100% of Splices at Same Location
≥ 1 1/2"	≥ 4"	#4	1'-11"
		#5	3'-0"
		#6	3'-7"

## APPENDIX 5-A (CONTINUED)

## MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

## ABUTMENTS:

**Abutment and Wingwall Vertical Bars**

Back face vertical bars are all spliced at the same location, so a Class B splice is used. See LRFD 5.11.3.1. Front face bars are conservatively assumed to act as tension reinforcement, so compressive development lengths are not used in splice length computations. Although all front face bars are spliced at the same location, excess reinforcement is provided, so a Class A splice is used.

Abutment and Wingwall Vertical Bar Lap Splice Lengths					
Concrete Cover to Bar Being Considered	Bar Size	Back Face Bar Spacing			Front Face Bar Spacing
		4"	5"	≥6"	≥6"
≥ 2"	#4	--	--	--	1'-6"
	#5	3'-0"	2'-5"	2'-5"	1'-10"
	#6	3'-7"	3'-7"	3'-7"	2'-9"
	#7	4'-6"	4'-2"	4'-2"	3'-2"
	#8	5'-11"	4'-9"	4'-9"	3'-8"
	#9	7'-6"	6'-0"	5'-10"	--
	#10	9'-6"	7'-7"	7'-2"	--
	#11	11'-8"	9'-4"	8'-8"	--
	#14	--	13'-5"	11'-10"	--

## APPENDIX 5-A (CONTINUED)

## MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

## ABUTMENTS: (cont'd)

**Abutment and Wingwall Horizontal Bars**

All horizontal bars are assumed to have more than 12" of concrete cast below. For abutments, horizontal bars are assumed to provide excess reinforcement, so a Class A splice is used. For long wingwalls on separate footings, horizontal bars become primary reinforcement, so a Class B splice is used.

Abutment and Wingwall Horizontal Bar Lap Splice Lengths				
Concrete Cover to Bar Being Considered	Bar Size	Abutment Horizontal Bar Spacing	Wingwall Horizontal Bar Spacing	
		$\geq 6"$	4"	$\geq 5"$
$\geq 2"$	#4	1'-11"	2'-6"	2'-6"
	#5	2'-5"	3'-4"	3'-1"
	#6	3'-1"	4'-0"	4'-0"
	#7	3'-7"	5'-1"	4'-8"
	#8	4'-1"	6'-8"	5'-4"

**APPENDIX 5-A (CONTINUED)**

**MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE**

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

**PIERS:**

**Pier Cap Top Longitudinal Bars**

All horizontal bars are assumed to have more than 12" of concrete cast below. For splices between columns where no more than 50% of the bars are spliced at the same location, a Class A splice is used. For all other cases, use a Class B splice.

<b>Pier Cap Top Longitudinal Bar Lap Splice Lengths</b>							
Concrete Cover to Bar Being Considered	Bar Size	All Splices Located Between Columns and $\leq 50\%$ of Bars Are Spliced at Same Location					
		Bar Spacing					
		4"	5"	5 1/2"	6"	$\geq 6 \frac{1}{2}"$	
$\geq 2 \frac{1}{2}"$	#5	2'-7"	2'-5"	2'-5"	2'-5"	2'-5"	
	#6	3'-1"	3'-1"	2'-10"	2'-10"	2'-10"	
	#7	3'-11"	3'-7"	3'-7"	3'-7"	3'-7"	
	#8	5'-2"	4'-1"	4'-1"	4'-1"	4'-1"	
	#9	6'-6"	5'-3"	4'-9"	4'-8"	4'-8"	
	#10	8'-3"	6'-7"	6'-0"	5'-6"	5'-6"	
	#11	10'-2"	8'-2"	7'-5"	6'-10"	6'-8"	
	#14	--	11'-9"	10'-8"	9'-9"	9'-1"	
			All Splices Located Between Columns and $> 50\%$ of Bars Are Spliced at Same Location				
			Bar Spacing				
			4"	5"	5 1/2"	6"	$\geq 6 \frac{1}{2}"$
		#5	3'-4"	3'-1"	3'-1"	3'-1"	3'-1"
		#6	4'-0"	4'-0"	3'-8"	3'-8"	3'-8"
		#7	5'-1"	4'-8"	4'-8"	4'-8"	4'-8"
		#8	6'-8"	5'-4"	5'-4"	5'-4"	5'-4"
		#9	8'-6"	6'-9"	6'-2"	6'-0"	6'-0"
		#10	10'-9"	8'-7"	7'-10"	7'-2"	7'-2"
		#11	13'-3"	10'-7"	9'-8"	8'-10"	8'-7"
		#14	--	15'-3"	13'-10"	12'-9"	11'-10"

**APPENDIX 5-A (CONTINUED)**

**MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE**

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

**PIERS: (cont'd)**

**Pier Cap Bottom Longitudinal Bars**

For splices over columns where no more than 50% of the bars are spliced at the same location, a Class A splice is used. For all other cases, use a Class B splice.

<b>Pier Cap Bottom Longitudinal Bar Lap Splice Lengths</b>							
Concrete Cover to Bar Being Considered	Bar Size	All Splices Located Over Columns and $\leq 50\%$ of Bars Are Spliced at Same Location					
		Bar Spacing					
		4"	5"	5 1/2"	6"	$\geq 6 \frac{1}{2}$ "	
$\geq 2 \frac{1}{2}$ "	#5	2'-3"	1'-10"	1'-10"	1'-10"	1'-10"	
	#6	2'-9"	2'-9"	2'-2"	2'-2"	2'-2"	
	#7	3'-6"	3'-2"	3'-2"	3'-2"	3'-2"	
	#8	4'-6"	3'-8"	3'-8"	3'-8"	3'-8"	
	#9	5'-9"	4'-7"	4'-2"	4'-1"	4'-1"	
	#10	7'-4"	5'-10"	5'-4"	4'-11"	4'-10"	
	#11	9'-0"	7'-2"	6'-7"	6'-0"	5'-10"	
	#14	--	10'-4"	9'-5"	8'-8"	8'-1"	
	Bar Size	All Splices Located Over Columns and $> 50\%$ of Bars Are Spliced at Same Location					
		Bar Spacing					
		4"	5"	5 1/2"	6"	$\geq 6 \frac{1}{2}$ "	
		#5	3'-0"	2'-5"	2'-5"	2'-5"	2'-5"
		#6	3'-7"	3'-7"	2'-10"	2'-10"	2'-10"
		#7	4'-6"	4'-2"	4'-2"	4'-2"	4'-2"
		#8	5'-11"	4'-9"	4'-9"	4'-9"	4'-9"
		#9	7'-6"	6'-0"	5'-5"	5'-4"	5'-4"
		#10	9'-6"	7'-7"	6'-11"	6'-4"	6'-4"
		#11	11'-8"	9'-4"	8'-6"	7'-10"	7'-7"
#14	--	13'-5"	12'-3"	11'-3"	10'-5"		

**APPENDIX 5-A (CONTINUED)**

**MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE**

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

**PIERS: (cont'd)**

**Other Pier Cap Longitudinal Bars Located on Side Faces of Pier Cap**

Longitudinal bars located on the side faces of pier caps (typically skin or shrinkage and temperature reinforcement) are assumed to have more than 12" of concrete cast below. For these bars, a Class B splice is used.

Lap Splice Lengths for Longitudinal Bars Located on Side Faces of Pier Cap		
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing $\geq 4"$
$\geq 2\ 1/2"$	#4	2'-6"
	#5	3'-4"
	#6	4'-0"
	#7	5'-1"

**Pier Column Vertical Bars**

For pier columns, all splices occur at the same location, so a Class B splice is used.

Pier Column Vertical Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing				
		4"	5"	5 1/2"	6"	$\geq 6\ 1/2"$
$\geq 2\ 3/8"$	#6	3'-7"	3'-7"	2'-10"	2'-10"	2'-10"
	#7	4'-6"	4'-2"	4'-2"	4'-2"	4'-2"
	#8	5'-11"	4'-9"	4'-9"	4'-9"	4'-9"
	#9	7'-6"	6'-0"	5'-5"	5'-4"	5'-4"
	#10	9'-6"	7'-7"	6'-11"	6'-4"	6'-4"
	#11	11'-8"	9'-4"	8'-6"	7'-10"	7'-7"
	#14	--	13'-5"	12'-3"	11'-3"	10'-5"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

SLAB SPANS:

Top Bars

This table applies to both top longitudinal and transverse bars. All bars are assumed to have more than 12" of concrete cast below. A Class B splice is used.

Top Longitudinal and Transverse Bar Lap Splice Lengths						
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing				
		4"	5"	6"	7"	≥ 8"
≥ 3"	#4	2'-6"	2'-6"	2'-6"	2'-6"	2'-6"
	#5	3'-4"	3'-1"	3'-1"	3'-1"	3'-1"
	#6	4'-0"	4'-0"	3'-8"	3'-8"	3'-8"
	#7	5'-1"	4'-8"	4'-8"	4'-4"	4'-4"
	#8	6'-8"	5'-4"	5'-4"	4'-11"	4'-11"
	#9	8'-6"	6'-9"	6'-0"	6'-0"	6'-0"
	#10	10'-9"	8'-7"	7'-2"	6'-9"	6'-9"
	#11	13'-3"	10'-7"	8'-10"	7'-7"	7'-6"
	#14	--	15'-3"	12'-9"	10'-11"	9'-11"

APPENDIX 5-A (CONTINUED)

MnDOT BRIDGE OFFICE REBAR LAP SPLICE GUIDE

- > Based on LRFD 5.11.2 and 5.11.5
- > Use of epoxy coated bars is assumed
- > Excess reinforcement factor  $\lambda_{er}$  is taken equal to 1.0

SLAB SPANS: (cont'd)

Bottom Bars

The table applies to both bottom longitudinal and transverse bars. A Class B splice is used.

Bottom Longitudinal and Transverse Bar Lap Splice Lengths			
Concrete Cover to Bar Being Considered	Bar Size	Bar Spacing	
		4"	≥ 5"
≥ 1 1/2"	#4	1'-11"	1'-11"
	#5	3'-0"	3'-0"
	#6	3'-7"	3'-7"
	#7	4'-8"	4'-8"
	#8	5'-11"	5'-11"
	#9	7'-6"	7'-3"
	#10	9'-6"	8'-11"
	#11	11'-8"	10'-7"
	#14	--	14'-4"

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## 6. STEEL STRUCTURES

Structural steel, in the form of rolled steel beams or welded plate girders, is used for bridge superstructures. In rare instances integral pier caps or substructures will be designed using steel. This section is intended to provide general design guidance and information on detailing practices. In addition, a design example for a two-span plate girder superstructure is included.

### 6.1 Materials

#### Structural Steels

Steel bridges are fabricated and constructed with steel elements that are produced at two different types of steel mills; shape mills and plate mills. In addition to different products, the grades of steel available from each type of mill differ slightly.

Shape mills produce bars, angles, tubes, pipes, channels, "W" sections (wide flange), "S" sections (American Standard), and piling that satisfy a variety of material specifications. Standard mill lengths available for these sections range from 30 to 60 feet. With sufficient quantities and sufficient lead time, longer lengths may be available. AISC's *Modern Steel Construction* yearly January issue provides information on different shapes available domestically from various mills. The designer shall check the availability of shapes before specifying their use in a structure.

Plate mills produce flat sections that are used to fabricate plate girders, connections, gusset plates, etc. Plate steel is also produced in a number of different material specifications. Larger plate mills have a width limitation of 150 inches. The maximum available plate length varies by mill and cross sectional dimensions of the plate.

The LRFD Specifications identify a number of steels that can be incorporated into bridge structures. They are identified in Tables 6.4.1-1 and 6.4.2-1 of the LRFD Specifications with both AASHTO and ASTM designations. Weathering steels have a "W" appended to the grade designation (e.g. 50W, 70W, 100W). Note that the AASHTO and ASTM designations are not identical.

Use weathering steel (Mn/DOT 3309, 3316, or 3317) for rolled beams, plate girders, and diaphragms on all steel bridges.

The AASHTO Specifications require additional tests (Charpy testing) to verify the toughness of the material. Mn/DOT Spec. 3308 requires this testing be conducted for steel incorporated into major structural components. Mn/DOT Spec. 2471.2 lists the specification numbers for

standard structural metals used in bridge projects. Structural steel of primary members shall satisfy the toughness requirements for Zone 3. Shape sections, other than pipe and tubing, are typically available in 36, 50, or 50W grades. Steel plate is available in 36, 50, and 50W grades. High Performance Steel (HPS) is available in Grades HPS50W and HPS70W. The typical steels for Mn/DOT bridge designs are Grade 50W and HPS70W. Hybrid sections may be the most cost effective, with the bottom flange in positive moment regions and both flanges near piers consisting of HPS70W, and all other steel 50W.

Additional information regarding steel shapes and plates may be found in *Properties of Bridge Steels*, Vol. I, Chapter 3, Highway Structures Design Handbook, May 1994.

The unit weight of steel is 0.490 kcf.

The coefficient of thermal expansion for steel is  $6.5 \times 10^{-6}$  in/in-°F.

#### [6.4.3]

#### **Bolts, Nuts, and Washers**

For most steel bridge applications ASTM A325 high strength bolts per Mn/DOT 3391.2B are to be used. The LRFD specifications also include ASTM A490 high strength bolts. Due to reduced availability and higher cost, A490 bolts should not be used without first consulting the Bridge Design Engineer. A490 bolts cannot be sold with plating, galvanizing, or mechanical zinc coating, so their use as field bolts is problematic.

For applications where strength is not the primary design consideration, ASTM A307 bolts per Mn/DOT Spec. 3391.2A may be used. See Mn/DOT Spec. 3391 for additional information on fasteners.

Additional fastener information may be found in *Mechanical Fasteners for Steel Bridges*, Vol. I, Chapter 4A, Highway Structures Design Handbook, April 1996.

Dimensional and weight information for bolts, nuts, and washers is provided in Appendix Figures 6-A1 through 6-A5.

#### **Shear Connectors (Stud Welded Fasteners)**

The material requirements for shear connectors are listed in Mn/DOT Spec. 3391. They shall satisfy ASTM material requirements, have a yield strength of 50 ksi, and an ultimate tensile strength of 60 ksi.

**Welds**

A variety of welding processes and materials are available to fabricators for different weld types. In most cases, designers need not concern themselves with the welding process selected by the fabricator.

Typically, only fillet welds and full penetration welds are permitted. Designs using partial penetration weld details can only be used with approval from the Fabrication Methods Engineer. Base weld designs on E70 filler material.

With the exception of pile splices, shear connectors, and railroad ballast plate splices, field welding is not used or permitted. Additional information on welding can be obtained from the Structural Metals Unit or Fabrication Methods Unit of the Bridge Office.

Additional references are the ANSI/AASHTO/AWS *Bridge Welding Code - D1.5*, and *Welding of Steel Bridges*, Vol. I, Chap 15, Highway Structures Design Handbook.

Appendix Figures 6-A6 through 6-A8 contain information on the proper construction of weld symbols and the proper application of the symbols to different types of details.

**Bearings**

Steel plates used in the fabrication of bearings shall meet Mn/DOT Spec. 3306, 3309, or 3310. Bearings made from castings shall satisfy ASTM A148, GR. 80-40.

**Paint Systems**

Use of weathering steel in Minnesota bridges has proven effective against continuous corrosion and section loss. Therefore, painting of weathering steel will only be considered under the following conditions:

- Near expansion joints (within 7 feet of the joint).
- Low level water crossings.
- Wide grade separations that may create tunnel-like conditions with persistent high humidity.
- Where warranted as an aesthetic treatment (limited to the outside surface and bottom flange of fascia beams).

All steel bridges will be reviewed during the preliminary design process to identify whether painting is required.

Preferred practice for painting is to have the primer applied in the fabrication shop and the intermediate and top or finish coat applied in the

field. Use the inorganic zinc-rich paint system (Mn/DOT Spec. 2479) for new steel designs.

For maintenance painting projects, use the organic zinc-rich paint system (Mn/DOT Spec.2478).

## **6.2 General Dimensions and Details**

### **[2.5.2.6.3]**

As a rule-of-thumb for the preliminary design of continuous structures, try a steel section depth of  $0.033L$ , where  $L$  is the span length. In no case should the steel section be less than  $0.0285L$ , unless approved by the Bridge Design Engineer. Typically a member taller than the minimum will be the most efficient. The most efficient depth of girder will vary with span and girder spacing. For large structures, perform a web depth study to arrive at the optimal girder height.

Locate field splices at or near points of dead load contraflexure.

Provide adequate spacing (2'-0" minimum) between field splices and diaphragm connection plates and stiffeners.

Identify "Area A" on the beam or girder plan sheets. "Area A" is the portion of the top flange that is in tension due to total dead load. Identifying the tension flange is important for a number of reasons:

- For complex bridge types or curved girders where a grid or 3-D analysis is needed, a bolted tab plate connection must be used to connect connection plates to tension flanges. A fillet weld is used for the connection to the compression flange. See Details B402 (bolted diaphragms), B407 (cross frame diaphragms), B408 (cross frame diaphragms for curved beams), and B410 (bolted stiffener to flange detail).
- During fabrication, identification of the tension flange is needed to complete the nondestructive testing requirements of MN/DOT Spec. 2471.
- During construction, the contractor is allowed per Mn/DOT Spec. 2402.3D to weld screed rail supports to the top flange of steel girders, except in "Area A".

For straight girders and those with slight curvature that meet the criteria given in LRFD Article 4.6.1.2.4b, connection plates may be connected to both the tension and compression flange using a fillet weld. Note that Standard Detail B407 allows for the use of either a welded or bolted connection. Because of its lower cost, the welded connection is preferred. The designer should first check the fatigue limit state at the diaphragm stiffener connections to determine if a welded detail is acceptable. If

stresses in the flange are too high to permit the welded detail, the designer should consider increasing the flange thickness to lower the stress range for fatigue or moving the diaphragm. Compare costs between the bolted option with initial flange thickness and the welded option with a thicker flange to determine the most economical option to show on the plan.

**[C6.7.4.1]**

The LRFD Specifications do not explicitly give a maximum diaphragm spacing as was previously given in the Standard Specifications. Diaphragms are used for bracing the compression flange and the diaphragm spacing is used to determine allowable compressive stresses.

Choose the diaphragm spacing in the positive moment area based on the maximum allowed for the bracing of the top compression flange during construction of the deck (typically 25 to 30 feet). In the negative moment area, the resistance of the bottom compression flange is based on the diaphragm spacing. The spacing in negative moment regions is usually 15 to 20 feet.

When choosing the distance from the centerline of bearing to the end of beam, use a minimum of nine times the web thickness.

For a steel superstructure, identify the following items on the framing plan:

- beam spacing
- staging
- distance between diaphragms along each beam
- type of diaphragms used in different locations
- centerline of bearing at piers and abutments
- Working Line and Working Points
- beam marks (B1, B2, etc.)
- type and location of bearings
- the location of intermediate stiffeners

The plate girder details shall identify plate sizes, length of plate segments, location of "Area A", spacing of shear studs, sole plate size, bearing and intermediate stiffener size, connection plate size, splice location and type, a table showing top of field splice elevations, and all pertinent notes. Standard notes are contained in Appendix 2-H of Section 2.

Structural steel plans and details must clearly describe the material to be used for each structural steel component. Even for projects where structural steel is paid for on a lump sum basis, provide informational

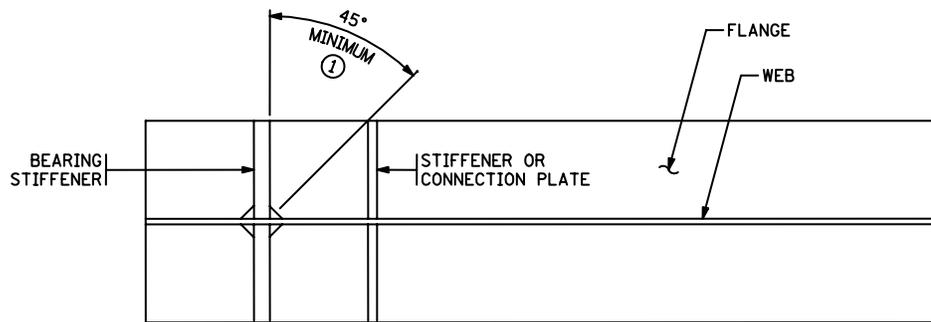
quantities in the plan set to quantify the amounts of different steels to be used. This is particularly true for projects with fracture critical members. Fracture critical members are fabricated to a higher quality standard to reduce the potential for defects and thus are more expensive. Do not specify members as fracture critical unless necessary and appropriate.

Structural steel quantities are computed by finding the weight of steel beams or girders, diaphragms, cross frames, and all other plates (e.g., sole and gusset plates). Increase the calculated weight by 1½% to account for the weight of steel for welds and bolt stick-through when computing structural steel quantities.

Designers should provide simple details that are easily fabricated and do not sacrifice the integrity of the bridge. Details that trap water or produce an environment that is conducive to corrosion should be avoided. In addition, details with inadequate clearances are difficult to fabricate and erect.

[AISC Steel Construction Manual, 13<sup>th</sup> Edition, Table 7-16]

The equipment used to weld and bolt steel pieces together requires room to operate. The *AISC Manual of Steel Construction* contains tables with minimum clearances for bolted connections. Figure 6.2.1 describes minimum clearances that must be provided for welded components.



PLAN VIEW

① WHEN DETAILING WELDED ATTACHMENT, ALLOW FOR A MINIMUM OPENING OF 45° FOR WELDING ACCESSIBILITY.

Figure 6.2.1

Show the type of weld to be used for each connection in the plans. The welding code (AWS-D1.5) specifies the minimum size of fillet welds. Identify the required weld size only if the minimum weld size does not have adequate capacity.

All connection details for lateral wind bracing systems shall be bolted.

For box pier caps and tub girders, make access holes and manholes through diaphragms as large as possible and locate for ease of passage. The minimum opening is 2'-0" x 2'-6". Provide an access door near each end of box piers for inspection purposes. Locate the door for ladder access off the roadway, if possible, and hinge the door to swing away from traffic. Place access doors in the side of the box where protected from superstructure runoff and in the bottom of the box where exposed. Use Mn/DOT Detail B942 for the door. Door frames shall be bolted to box.

Where single conduits pass through steel diaphragms and require a passage hole with a diameter greater than 3 inches, reinforce the opening with a section of pipe or curved steel plate.

### ***6.3 General Design Philosophy***

In general, structural steel superstructures are shallower and lighter than concrete superstructures. In addition to long span and specialty structures, steel superstructures should be considered where foundations are expensive or where a change in superstructure height has significant cost implications on the approaches.

Design girders to be composite with the concrete deck throughout the entire girder length. Provide shear connectors, in the form of shear studs, in both positive and negative moment areas and over field splices.

Stools are used with steel superstructures to provide a construction tolerance for the profile of the deck. The stools shall have vertical edges that are flush with the edges of the top flange. For plate girders the stool is defined as the distance between the bottom of the deck and the top of the web. For rolled beams the stool height is defined as the distance between the bottom of the deck and the bottom of the top flange. Stool heights are to be given at the centerline of the beam. The minimum height for the concrete portion of the stool is 1½ inches. This minimum is measured at the edge of the flange taking into account the cross slope of the deck. At field splices check that the top plates do not penetrate the bottom of the deck.

During design, it may be assumed that the dead load of the steel beam or girder is 15% larger than that computed using only the flanges and web. This is a reasonable estimate for the weight of stiffeners, diaphragms or cross frames, and connections.

Use of the moment redistribution provisions of LRFD Appendix B is not allowed.

The maximum nominal flexural resistance  $F_n$  allowed by Mn/DOT for design of steel beams is equal to  $F_y$ . Do not exceed the moment at first yield.

Use a limited number of thicknesses when sizing stiffeners and connection plates. To permit two lines of bolts, connection plates must be a minimum of 7" in width.

For steel superstructures with uplift at the abutments, the end diaphragms and/or counterweight shall be cast prior to deck construction and the deck shall be cast beginning at the abutment with the greatest uplift. Temporary tie-downs at the abutments may be necessary for the deck pour.

Provide bent plate diaphragms (Mn/DOT Detail B402) for the following cases:

- rolled beam superstructures
- plate girders with depths less than 40 inches
- beam depth to lateral spacing ratio less than 0.40

In other cases, use cross frame diaphragms (Mn/DOT Detail B407).

Railroad bridges designed in accordance with the AREMA Specifications may have slightly different criteria than AASHTO for high strength bolts, pin bolts, and welding. Check with the railroad in question for specific criteria.

### **6.3.1 Shear Connectors** **[6.10.7.4.1]**

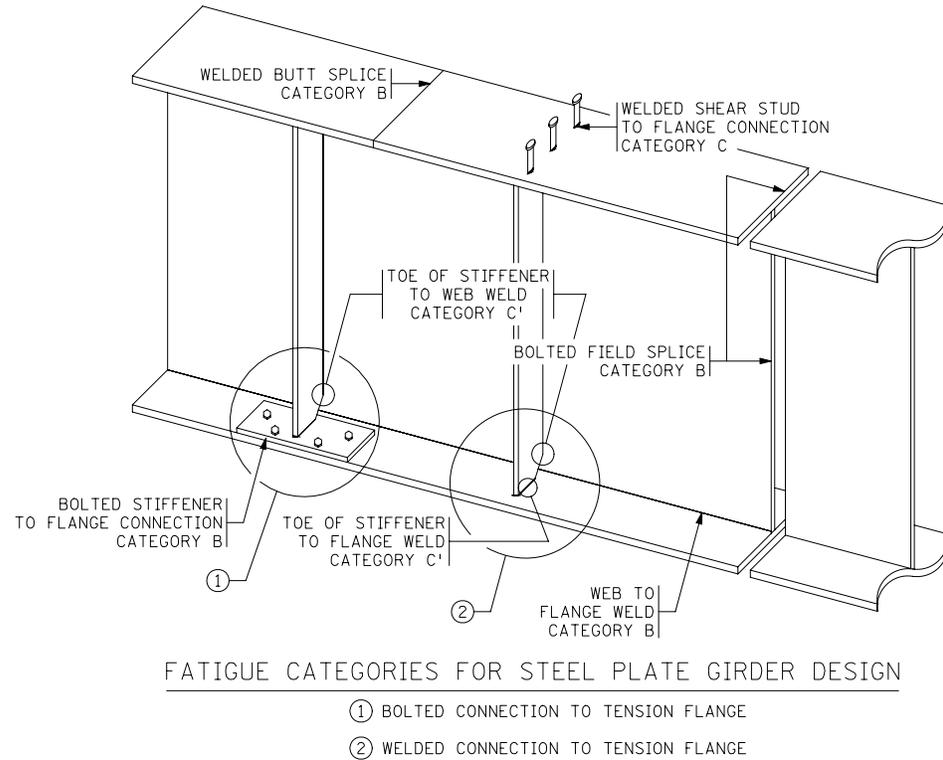
Provide  $\frac{7}{8}$  inch diameter stud connectors that extend a minimum of 2 inches above the bottom of the deck and a maximum of 3 inches below the top of the deck. Studs must be applied in the field after girder erection.

### **6.3.2 Fatigue**

Fatigue cracks are generally classified as either load induced or displacement induced. Load and stress limits are placed on members to minimize load induced fatigue cracks from forming. Proper detailing practices are used to prevent displacement induced fatigue cracks.

Designers must check connections for fatigue resistance. For all Trunk Highway bridges, check details for an infinite fatigue life level regardless of ADT level.

Figure 6.3.2.1 identifies the appropriate fatigue category to be used for typical plate girder connections. Check all fatigue categories that apply. For discussion of "Area A" and welding vs. bolting connection plates to the tension flange, see Article 6.2 of this manual.



**Figure 6.3.2.1**

**[6.10.8.1.1]**

Detailing practices that prevent displacement induced fatigue cracks from forming include coping stiffeners and terminating welds slightly before reaching the end of an element. Tops and bottoms of transverse stiffeners and connection plates are typically coped  $1\frac{1}{2}$  inches from face of web and  $2\frac{1}{2}$  inches from face of flange. (See Mn/DOT Detail B411.)

**6.3.3 Deflections**

To ensure that bridges are constructed with a proper vertical profile, the deflections associated with selfweight, deck placement, and composite superstructure dead loads shall be presented in the plan set. Split the dead load deflection into two categories: selfweight (including diaphragms), and dead load due to deck and all superimposed loads (excluding future wearing course). Display deflections in feet with a precision of three decimal places.

Live load deflection shall be limited to  $L/800$  for typical bridges and  $L/1000$  for bridges carrying pedestrians. Calculate the live load distribution for deflection by taking the number of lanes times the multiple presence factor and divide by the number of beams. The multiple presence factor used with this calculation shall not be less than 0.85.

#### **6.3.4 Camber**

For most steel bridges, camber is fabricated into the beam to match the profile grade and offset the deflections due to applied dead loads. In some cases, residual camber (extra camber added for architectural reasons) is also added to eliminate the possible appearance of a sag in a span.

For rolled beams introducing camber can be a relatively expensive operation. It is usually accomplished with cold bending and/or with heat straightening techniques. Camber rolled beams for bridges only if the dead load deflection exceeds the maximum mill tolerance for camber, which is equal to  $1/8$  inch per 10 feet of length. If the deflection is less, state that the beam shall be placed "natural camber up".

Plate girders shall always be cambered. Vertical cambers are introduced by cutting the web plates into the desired profile. Horizontal curvature is introduced (if necessary) by cutting flange plates with the proper horizontal shape. During fabrication, the web and flanges are attached to each other to produce a member with the proper geometric characteristics.

Camber for vertical curvature, anticipated dead load deflections, and residual camber (if required). Do not include the deflection due to future wearing course (FWC). Provide residual camber only in girders with straight grades with lengths in excess of 100 feet. Use approximately  $1\frac{1}{2}$  inches of residual camber for a 100 foot span. Increase the residual camber by  $1/8$  inch for each 10 foot change in span length. Use a maximum residual camber of  $2\frac{1}{2}$  inches.

Choose a stool height that will be constant throughout the length of the girder for girders without residual camber. For girders where residual camber is used, the stool height will vary. For these situations, the stool height will have its largest value at substructure locations and smaller values near midspan. In no case is the thickness of concrete in the stool to be less than  $1\frac{1}{2}$  inches.

The following procedure may be used to develop a camber diagram:

- 1) The camber curve, a line located at the bottom of top flange for rolled beams and at the top of web for plate girders, defines the cambered shape of the member. Geometric camber, dead load camber, and residual camber (if required) are the components that make up the camber curve. Start by determining the geometric camber profile due to the vertical geometry of the roadway. To do this:
  - Calculate profile grade elevations at tenth points along the member as well as at field splice and/or point of contraflexure locations.
  - Calculate top of deck elevations at centerline of member by adjusting for cross slopes and offset from profile grade.
  - Calculate geometric camber profile by subtracting the deck thickness and stool height.
- 2) Determine total dead load (minus future wearing course) deflections for the member. Downward deflections are considered negative and upward deflections are considered positive. The dead load camber profile is the opposite sign (downward +, upward -) of the total dead load deflections.
- 3) If there is no residual camber, add the dead load camber profile to the geometric camber profile to get the final camber curve.
- 4) If residual camber is required, calculate residual camber profile assuming a maximum value at midspan and parabolic distribution over the rest of the span. Then, add the residual camber profile and the dead load camber profile to the geometric profile to get the final camber curve.
- 5) Establish a horizontal line at the substructure centerline of bearing with the lowest camber curve elevation.
- 6) Establish chord lines, which are defined as straight lines between the end of each beam segment at the camber curve.
- 7) Determine all vertical and horizontal dimensions to be entered on camber diagram to the nearest  $\frac{1}{8}$  inch. These include:
  - Dimensions from horizontal line to camber curve at all support points, field splices, and contraflexure points.
  - Dimensions from horizontal line to camber curve for at least three points intermediate to those in the first bullet for each curved section. Locate one of these points at the point of maximum camber within the curved section.
  - Maximum camber dimensions from chord line to camber curve for each curved section.
  - Span lengths, segment lengths, and dimensions from supports to field splices and contraflexure points.
  - Locations of camber dimensions along the member.

Refer to Figure 6.3.4.1 for a sample camber diagram.



**6.4 Rolled Beams**

Rolled beams may be difficult to obtain in lengths over 90 feet. Check with the Fabrication Methods Unit prior to incorporating beams with lengths over 90 feet into a design.

Use rolled beam sections with a minimum flange width of 14 inches. This will allow four lines of bolts to be used in the field splice design.

**6.5 Plate Girders**

For shipping purposes, limit the length between field splices to 145 feet.

Select plate thickness in  $\frac{1}{16}$  inch increments for thicknesses up to 1 inch. For thicknesses between 1 and  $2\frac{1}{2}$  inches, use  $\frac{1}{8}$  inch increments. Use  $\frac{1}{4}$  inch increments for thicknesses between  $2\frac{1}{2}$  and 4 inches.

In general, additional web thickness increases shear capacity. An increase in web height or flange area increases moment capacity and reduces live load deflections.

In general, follow these guidelines in plate size selection for plate girders:

**Flanges****[C6.10.3.4]**

For plate girder flanges, the minimum size is  $\frac{3}{4}$ " x 14". The 14 inch flange permits four lines of bolts for field splices. Limit the ratio of segment length to top flange width to 85 for stability during shipping and erection.

The change in flange area at butt weld splices shall not exceed 100%. In general, it is economical to provide a butt splice if 1000 to 1200 pounds or more of steel can be saved. Further discussion on this issue can be found in Article 1.5.2 of *G12.1 - 2003 Guidelines for Design for Constructibility*.

Where practical, keep the bottom flange at a constant width over the entire girder length. Top flanges should be kept at a constant width within each field piece. Use a 24 inch radius for the taper detail where flanges need to be tapered in width. When changing the flange width and thickness at a butt splice, first taper the width and then taper the thickness. If changing the top flange width at a field splice, do not taper the flange width.

**[6.13.6.1.5]**

When thick fill plates are required at field splices, additional rows of bolts will be required to transfer the force to the member.

**Web**

For web plates the minimum thickness is  $\frac{1}{2}$  inch. The  $\frac{1}{2}$  inch web reduces the potential for warping during fabrication. Select maximum web height while still meeting clearance requirements.

When choosing a web thickness, first determine the thickness at which no intermediate stiffeners are required for shear. Reduce the web thickness in  $\frac{1}{16}$  inch increments and calculate the weight of web steel saved per stiffener added. Generally, it is economical to reduce the web thickness when the weight of web steel saved per stiffener added is greater than 1000 pounds.

Web thickness changes are allowed at field splices. For a thickness change of  $\frac{1}{16}$  inch, detail the web splice with a 16 gauge fill plate on one side of the web only.

Longitudinal stiffeners should only be considered for girders over 84 inches deep. Terminate longitudinal stiffeners at a low stress point with a fatigue resistant detail. Generally, detail longitudinal stiffeners as continuous through transverse and bearing stiffeners.

**6.5.1 High Performance Steel Girders**

Use of High Performance Steel (HPS) Grade HPS 70W ( $F_y = 70$  ksi) may be an economical alternative to 50 ksi steel. Typically, a hybrid design that utilizes HPS steel for the bottom flange in positive moment areas and both flanges in negative moment areas is most economical.

**6.6 Horizontally Curved Steel Girders**

The 2005 Interim Specifications unified Section 6 to include both straight and horizontally curved steel bridges. This article highlights some of the issues particular to curved steel design.

**[6.10.6-6.10.8]****Flexure**

The preliminary depth and girder spacing shown in the Preliminary Plan is determined using a straight line girder analysis with a maximum bending stress limit of  $0.85F_y$ . Use the Preliminary Plan to develop a framing plan for review with the Design Unit Leader and Bridge Design Engineer. Once the framing plan has been approved, analyze the bridge as a system using an appropriate structural analysis program.

Design considerations unique to horizontally curved steel girders include:

**[4.6.1.2.4b]**

- The span, radius, and skew of the girder determine whether the curvature must be considered in the analysis.
- Curved steel girders are always considered noncompact in the positive moment region. Therefore the maximum nominal bending stress is  $F_y$ .
- Use of Appendix A or Appendix B is not allowed.
- Lateral flange bending stresses due to torsion must be taken into account. As a result, curved steel plate girder bridges usually have wider flanges than straight steel bridges.
- Horizontal curvature causes a variable load distribution that increases from inside to outside of the curve. Theoretically, flange and web sizes could be different for each girder. The designer must consider the economic benefits associated with grouping plate sizes. In other words, consider grouping the girders, using identical flange sizes for the fascia and first interior beam, the second and third interior beam, etc. Also consider carefully whether to incrementally increase the web depth from inside to outside of the curve. This practice may cause the outside beams to become too stiff, drawing too much moment to the outside fascia beam.

**[6.10.9]**

**Shear**

Web shear capacity is treated the same for both straight and horizontally curved steel girders.

**[6.7.4]**

**Diaphragms**

Diaphragms are considered primary structural members in curved bridges. Intermediate diaphragms may be either cross frame or bent plate type (Detail B408 or B402).

Use the following guidance for design and detailing of diaphragms:

- Maximum diaphragm spacing given in the *LRFD Bridge Design Specifications* is the lesser of  $R/10$  or 30 feet for curved steel girders. Mn/DOT more conservatively limits the diaphragm spacing to 25 feet.
- Design diaphragms and their connections for the factored forces and moments determined by analysis. Unlike beam splices, design for higher loads is not required. If the analysis software allows, include the deck in the analysis model to reduce diaphragm stresses.

**[6.13.1]**

- Generally, provide radial lines of diaphragms which extend across the entire width of the bridge. Diaphragms may be discontinued near the obtuse corner of skewed abutments or over skewed piers to provide flexibility between supports and adjacent members.
- Place cross frame members such that their lines of action intersect at the center of gravity of the bolt group or produce the smallest possible moment on the bolt group that connects the gusset plate to the connection plate. If the lines of action must move, balance the forces such that moments on the connection are minimized.
- Due to the high cost of WT shapes, use angles for all cross frame members. Place all angles with the vertical leg projection down to prevent debris from collecting in the angle.
- Due to the high cost of bolting, connect angle members to the gusset plate with welds. Also, use a connection at the intersection of cross frame diaphragm diagonals. Use all-around welds to prevent moisture and debris from collecting between members. Consider weld details for diaphragms as Fatigue Category E, but allow a fatigue overstress of 15% per Mn/DOT policy.
- Design welds for the shear force and moment (caused by the member eccentricity) in the plane of the weld. Neglect the out-of-plane moment caused by the vertical eccentricity of the angle. When designing welds, check if the minimum weld size is adequate and increase as needed. The weld length may also be increased depending on the connection geometry.
- Design the gusset plate for the moment induced by the connected members. A reduction in the height of the gusset plate may be required as the full gusset plate may not aid in resisting the applied forces. Also, check the axial stress induced in the gusset plate by the connected members.
- Design cross frame members for factored axial forces. When in compression, slenderness of the members shall be computed using an effective length factor  $K = 1.0$ .

**[4.6.2.5]**

**[6.10.10]****Miscellaneous**

The design of shear connectors in horizontally curved steel beams accounts for shear forces produced in the longitudinal direction by beam bending, and shear forces produced in the radial direction by the cross frames. The available curved girder analysis programs do not clearly specify the direction of cross frame forces under fatigue loadings. Since this makes it difficult to obtain an accurate net fatigue force range, cross frame forces can conservatively be added for simplicity.

Design curved steel girder splices for vertical bending, lateral bending and shear. Composite section properties shall be used.

Calculate camber based on dead load deflection, vertical curvature, and residual camber (if required). Compute deflections assuming the deck is poured in a 7 inch lift followed by a 2 inch wearing course.

For shipping purposes, limit the length between field splices to 100 feet when the offset from the chord connecting the ends is between 3 and 6 feet. A shipping length of 145 feet can be used when the offset is less than 3 feet. Check with the Fabrication Methods Engineer for specific situations.

For unusual circumstances, consider requiring the contractor to use erection shoring. Prior to using this design assumption, discuss the project with the Bridge Design Engineer and the Regional Bridge Construction Engineer.

Full assembly should be considered for curved steel superstructures. Discuss use of full assembly vs. special assembly with the Fabrication Methods Engineer or Structural Metals Engineer before specifying on the Plan.

**6.7 Box or Tub  
Girders**

Box or tub girders have rarely been used in Minnesota, but may be an economical choice for longer span bridges. Typically, they are trapezoidal in shape, with two top flanges, two webs, and a single wide bottom flange. The top flanges have shear connectors attached to them that are used to develop composite action with a cast-in-place deck. Once the deck is in place the closed shape of the cross section is effective in carrying torsional loads in addition to flexural loads.

Ensure that the structure has adequate capacity prior to the development of composite action with the deck. The lateral bracing system for the top flanges must be considered during construction.

**6.8 Bolted Connections and Splices**

Bolted connections are used mainly in field splices, diaphragms, and metal railings.

Check details to ensure that there are no bolting access or assembly problems.

**Splices**

Use  $\frac{7}{8}$  inch diameter ASTM A325 bolts. The standard bolt pattern is a 3 inch grid with edge distances of  $1\frac{1}{2}$  inch.

Use a maximum gap equal to  $\frac{3}{8}$  inch between the ends of spliced beams. Provide a minimum of 3 inches from the inside of the inside flange splice plates to the center of the first row of bolts in the web splice.

The change in flange area at bolted splices shall not exceed 100%. The splice plates must be of the same steel as the elements being connected. The minimum thickness of splice plates is  $\frac{5}{16}$  inches.

Design bolted field splices as slip-critical connections. Assume that a Class B surface coating or condition is available for slip resistance (Slip Coefficient 0.50).

Include the standard plan note concerning field splice elevations on the detailed drawings.

Where splice plates are  $\frac{3}{8}$  inch thick or greater, connections may be designed assuming threads are excluded from the shear plane.

**6.9 Two-Span  
Plate Girder Design  
Example**

This example illustrates the design of a two-span welded plate girder bridge with a 20 degree skew. The bridge is on a constant grade of 1.5% and has two equal spans of 175'-0". Mn/DOT standard details and drawings for diaphragms (B402 and B407) and railings (Fig. 5-397.117) should be referenced when reviewing this example. This example includes the detail design of a typical interior girder at the critical sections in flexure and shear for AASHTO HL-93 loading. Design of the stiffeners, end diaphragm, shear connectors, and field splice is also included. Fatigue is checked at critical locations.

The superstructure consists of five girders spaced at 11'-4" centers. Girders are designed to act compositely with a 9<sup>1</sup>/<sub>2</sub> inch deck. A 1/2 inch of wear is assumed and a deck thickness of 9 inches is used for composite section properties.

**A. Materials and  
Geometry**

The following material and geometric parameters are used in this example:

Concrete (deck and overlay)

Dead load unit weight  $w_c = 0.150$  kcf

Compressive strength  $f'_c = 4$  ksi

Elastic modulus  $E_c = 3644$  ksi

Steel

Dead load unit weight  $w_{st} = 0.490$  kcf

Yield strength  $F_y = 50$  ksi

Tensile strength  $F_u = 70$  ksi

Elastic modulus  $E_s = 29,000$  ksi

**[6.10.1.1.1b]**

Composite Section Properties

Short-term modular ratio =  $n = 8$

Long-term modular ratio =  $3n = 24$

Average Daily Truck Traffic 2000 per day

The overall geometry for the example is presented in Figures 6.9.1 and 6.9.2. Girder geometry is presented in the next section (see Figures 6.9.3 and 6.9.4) where section properties are assembled.

A typical section for the bridge is shown in Figure 6.9.1. The deck is supported on five lines of girders. The girders are spaced on 11'-4" centers and the roadway is 48'-0" wide (two 12'-0" traffic lanes and two 12'-0" shoulders). A Type F-rail is provided on each side of the bridge.

The framing of the superstructure is presented in Figure 6.9.2. The structure has a 20 degree skew. Due to the symmetric span arrangement, only a half-framing plan is provided. Rolled beam end diaphragms are located at the abutments. Cross frames are used for interior diaphragms.

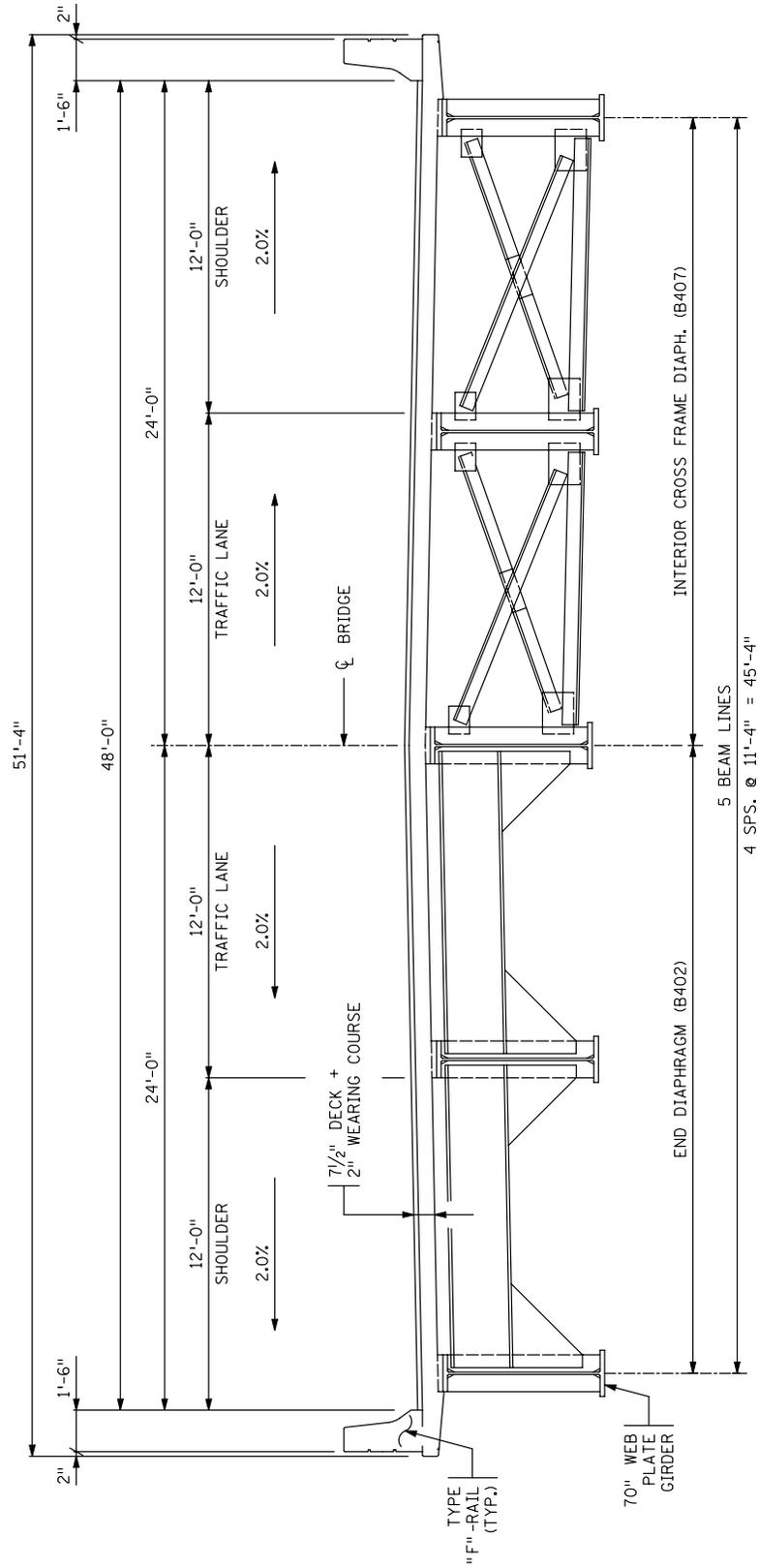
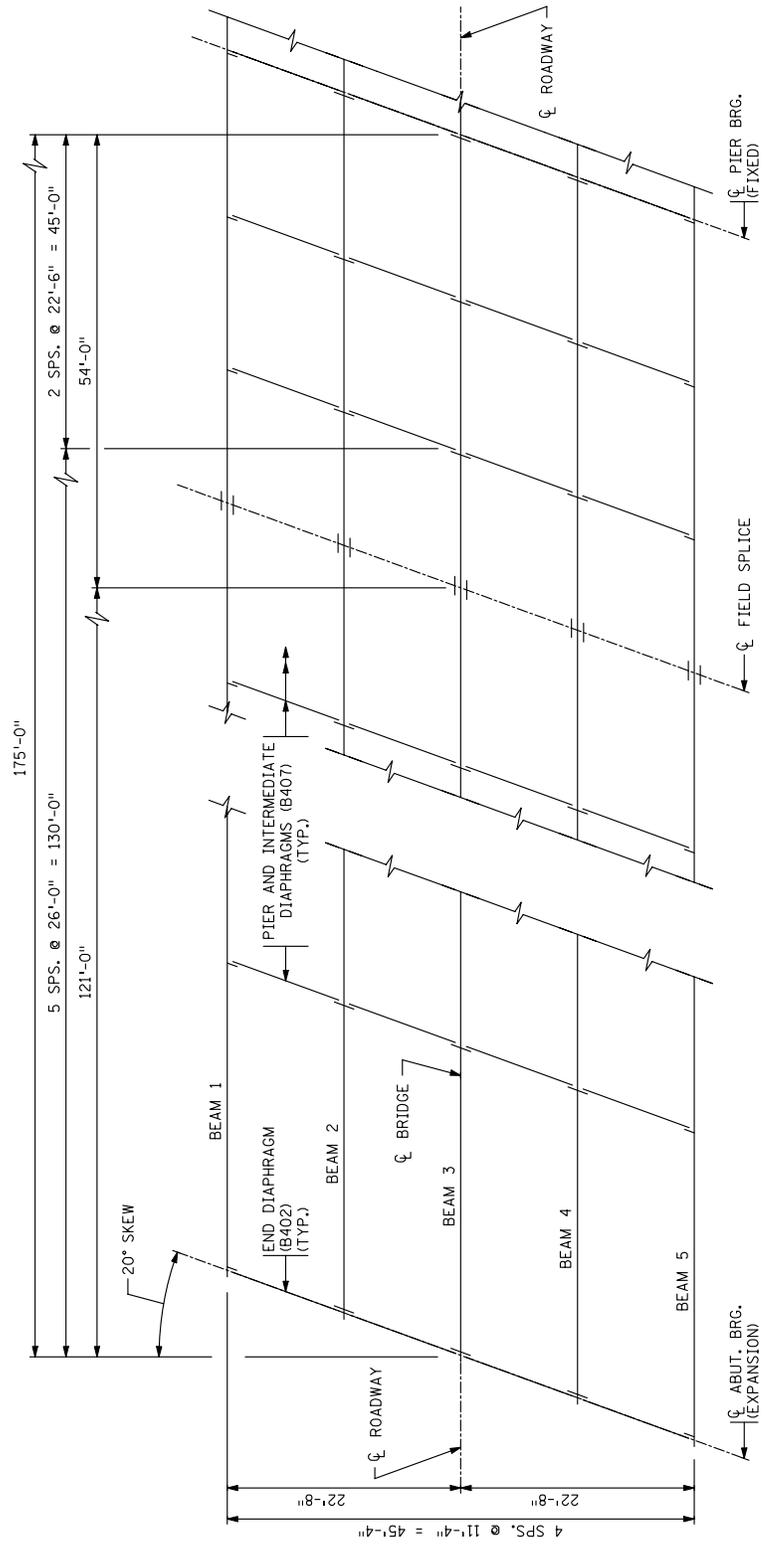


Figure 6.9.1



**Figure 6.9.2**  
**Partial Framing Plan**

**B. Determine Cross Section Properties****Non-Composite Section Properties**

The minimum depth of the steel girder (see Section 6.2 of this manual) in a continuous span is  $0.0285L$ . Mn/DOT typically considers a preliminary depth of  $0.033L$ .

For  $L = 175$  ft:

$$0.0285 \cdot L = 59.9 \text{ in}$$

$$0.033 \cdot L = 69.3 \text{ in}$$

A member deeper than the minimum is usually the most economical. Adequate clearance is assumed available for the example, so try a 70 inch deep web.

**[6.10.2.1.1]**

Webs without longitudinal stiffeners must be proportioned such that:

$$\frac{D}{t_w} \leq 150$$

Then, the minimum web thickness  $t_w$  is:

$$t_w \geq \frac{D}{150} = \frac{70}{150} = 0.47 \text{ in}$$

Section 6.5 of this manual requires a minimum web thickness of  $1/2$  inch.

Try a web thickness of  $5/8$  inch.

**[6.10.2.2]**

The minimum flange width  $b_f$  and flange thickness  $t_f$  is:

$$b_f \geq \frac{D}{6} = \frac{70}{6} = 11.7 \text{ in}$$

$$t_f \geq 1.1t_w = 1.1(0.625) = 0.6875 \text{ in}$$

The minimum flange size specified by Section 6.5 of this manual is  $3/4$ " x 14".

**[C6.10.3.4]**

For stability during shipping and erection the minimum compression flange width  $b_{fc}$  is:

$$b_{fc} \geq \frac{L}{85} = \frac{121 \cdot (12)}{85} = 17.1 \text{ in}$$

Based on experience and the minimum proportions given above, preliminary web and flange plate sizes were chosen and are shown in Figure 6.9.3. The girder is symmetric about the pier with a 1" x 20" top flange and a 1<sup>1</sup>/<sub>4</sub>" x 22" bottom flange in the positive moment region. In the negative moment region, the top flange is 1<sup>3</sup>/<sub>4</sub>" x 20" near the field splice and 3<sup>1</sup>/<sub>4</sub>" x 20" over the pier. The bottom flange is 1<sup>3</sup>/<sub>4</sub>" x 22" near the field splice and 3<sup>1</sup>/<sub>4</sub>" x 22" over the pier. For the web, a <sup>5</sup>/<sub>8</sub>" x 70" plate is used throughout.

**[6.10.2.2]**

Two additional flange proportion checks are required:

$$\frac{b_f}{2t_f} \leq 12.0 \quad \text{and} \quad 0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10.0$$

Make each check using the most critical section:

For 1" x 20" flange,  $\frac{b_f}{2t_f} = \frac{20}{2 \cdot (1.0)} = 10.0 < 12.0$  OK

For Design Section 1,

$$I_{yc} = \frac{1.00 \cdot (20)^3}{12} = 666.7 \text{ in}^4$$

$$I_{yt} = \frac{1.25 \cdot (22)^3}{12} = 1109.2 \text{ in}^4$$

$$\frac{I_{yc}}{I_{yt}} = \frac{666.7}{1109.2} = 0.60 \quad 0.1 < 0.60 < 10.0 \quad \underline{\text{OK}}$$

The non-composite section properties of the girder are provided in Table 6.9.1.

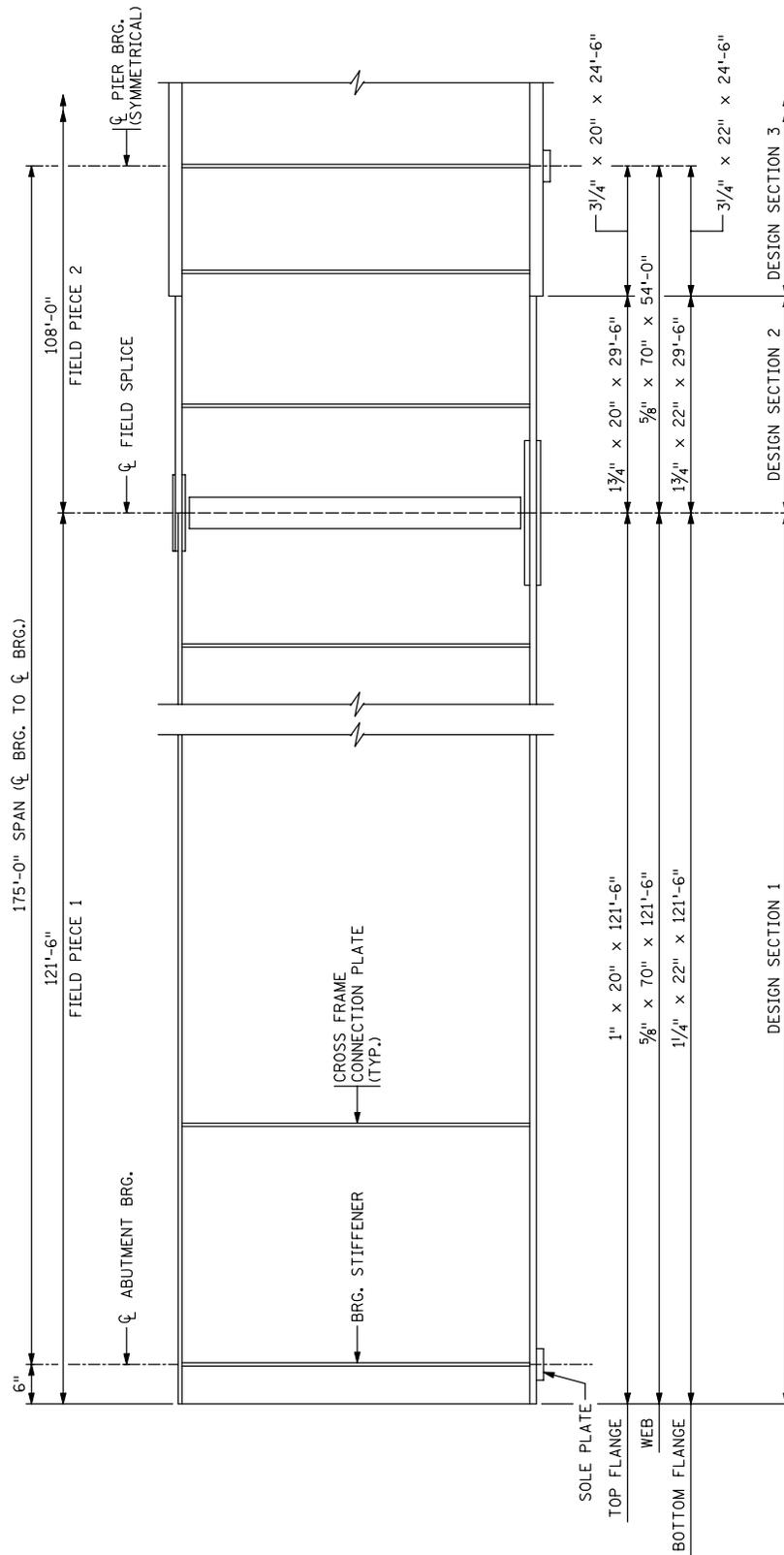


Figure 6.9.3  
Preliminary Beam Layout – Half Elevation

**Table 6.9.1**  
**Non-Composite Section Properties**

Parameter	Design Section 1 *	Design Section 2 **	Design Section 3 ***
$d_{nc}$ (in)	72.25	73.50	76.50
$A$ (in <sup>2</sup> )	91.25	117.25	180.25
$I$ (in <sup>4</sup> )	77,179	112,345	200,770
$y_t$ (in)	38.96	37.82	39.57
$y_b$ (in)	33.29	35.68	36.93
$S_t$ (in <sup>3</sup> )	1981	2970	5074
$S_b$ (in <sup>3</sup> )	2318	3149	5437

\* Design Section 1 is from abutment to field splice

\*\* Design Section 2 is from field splice to flange butt splice

\*\*\* Design Section 3 is section over pier

**Effective Flange Width**

For simplicity, and in order to be conservative, the beams are designed assuming the full 9<sup>1</sup>/<sub>2</sub> inches of deck thickness is placed in a single pour instead of the actual two pours.

For section property computations the deck thickness is reduced by <sup>1</sup>/<sub>2</sub> inch to account for wear.

**[4.6.2.6]**

The width of deck  $b_{eff}$  assumed to act compositely with the girder and resist external loads is the smallest of three values:

$$b_{eff} = 0.25 \cdot (\text{Effective span length}) = 0.25 \cdot (0.7 \cdot 175) \cdot 12 = 368 \text{ in}$$

or

$$b_{eff} = 12 \cdot (\text{Deck thickness}) + 0.5 \cdot (\text{Top flange width})$$

$$= 12 \cdot 9 + 0.5 \cdot 20 = 118 \text{ in} \quad \text{GOVERNS}$$

or

$$b_{eff} = \text{Average beam spacing} = 136 \text{ in}$$

**Positive Moment Region Composite Section Properties**

Using the modular ratios provided earlier ( $n = 8$ ,  $3n = 24$ ) results in:

**[6.10.1.1.1b]**

Transformed  $b_{eff_n}$  for transient, short-term loads

$$b_{eff_n} = \frac{118}{8} = 14.75 \text{ in}$$

Transformed  $b_{\text{eff}_{3n}}$  for permanent, long-term loads

$$b_{\text{eff}_{3n}} = \frac{118}{24} = 4.92 \text{ in}$$

Choose a stool height for the girder, which is defined as the distance from the bottom of the deck to the top of the web. Because the top flange thickness varies along the girder length, the concrete portion of the stool will vary. The minimum required thickness of the concrete portion of the stool is  $1\frac{1}{2}$ " at the edge of the flange. Therefore, the stool height is dependent on the thickest top flange plate, which is located at the pier.

The largest top flange plate is  $3\frac{1}{4}$ " x 20" and the deck cross slope is 2%.

Then, the minimum required concrete portion of the stool height along the girder centerline is:

$$\text{stool}_{\text{min conc}} = 1.50 + 0.02 \cdot 0.5 \cdot 20 = 1.70 \text{ in} \quad \text{Use} = 1.75 \text{ in}$$

The minimum required total stool height along the girder centerline at the pier is:

$$\text{stool}_{\text{min pier}} = 3.25 + 1.75 = 5.0 \text{ in}$$

This bridge is on a straight grade, so residual camber is required, which "eats" into the stool at midspan. Therefore, check the minimum required stool height at midspan also to see if it governs.

The required residual camber is:

$$\text{camber}_{\text{res}} = 1.50 + \left( \frac{175 - 100}{10} \right) \cdot 0.125 = 2.44 \text{ in}$$

$$\text{Use camber}_{\text{res}} = 2.50 \text{ in}$$

The top flange plate at midspan is 1" x 20". Then, the minimum required total stool height along the girder centerline at midspan is:

$$\text{stool}_{\text{min midspan}} = 1.00 + 1.75 + 2.50 = 5.25 \text{ in} \quad \underline{\text{GOVERNS}}$$

Choose a stool height of 5.25 inches.

For calculation of the girder section properties, use the minimum concrete stool height  $t_{\text{cstool}}$  equal to 1.75 inches.

See Figure 6.9.4 and Table 6.9.2 for the composite sections and computed properties used for design in the positive moment region. Note that only Design Sections 1 and 2 fall in the positive moment region.

### Negative Moment Region Composite Section Properties

#### [6.10.1.1.1c]

For negative moment regions, the section assumed effective in resisting external loads is the steel girder section plus the reinforcement within an effective width of the slab.

#### [6.10.1.7]

In negative moment regions, the longitudinal reinforcing steel in the deck is approximately 1% of the area of the deck. Two thirds of this steel is to be placed in the top mat of reinforcement. Referring to Figure 9.2.1.7, the area of steel within the effective flange width is:

Top mat: #16 bars @ 18" with 2- #19 bars in between

$$A_{\text{stop}} = (0.31 + 2 \cdot 0.44) \cdot \left( \frac{118}{18} \right) = 7.80 \text{ in}^2$$

Bottom mat: #13 bars @ 6"

$$A_{\text{sbot}} = 0.20 \cdot \left( \frac{118}{6} \right) = 3.93 \text{ in}^2$$

The top mat is located 3.50 inches from the top of the deck (based on 3 inches clear,  $\frac{1}{2}$  inch wear, and #16 transverse bars) and the bottom mat is located 1.88 inches from the bottom (based on 1 inch clear, and #16 transverse bars). See Figure 6.9.4 and Table 6.9.2 for the composite sections and computed properties used for design in the negative moment region.

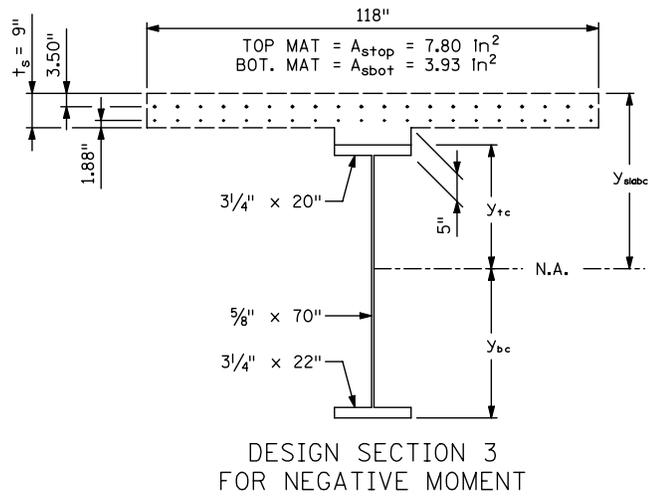
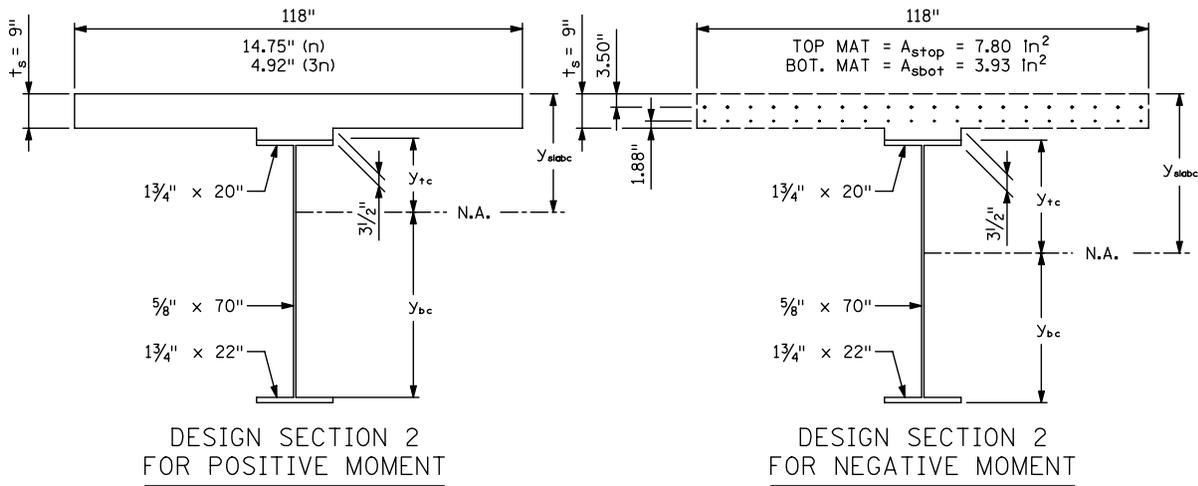
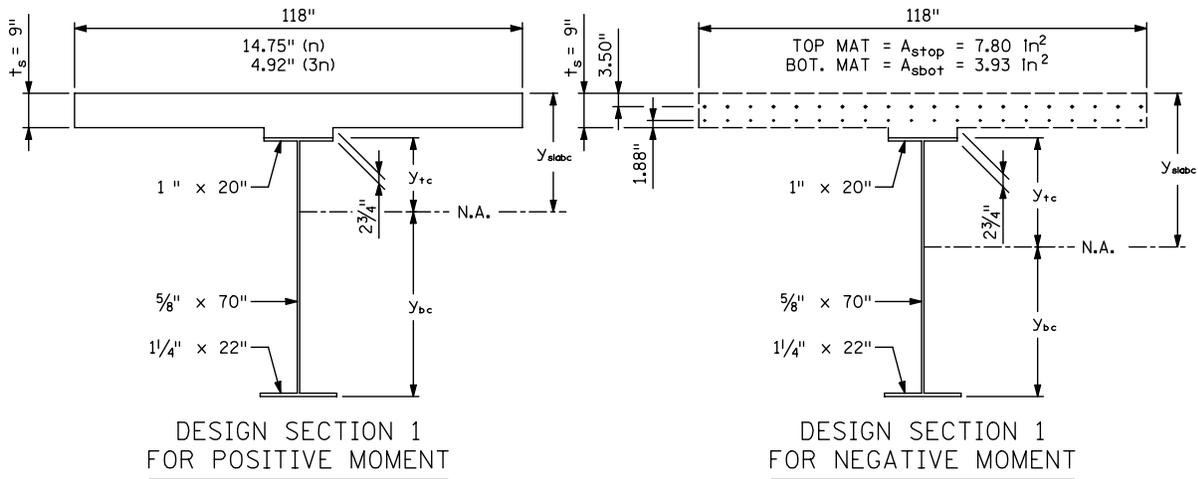


Figure 6.9.4

**Table 6.9.2  
Composite Section Properties for Design**

Parameter	Design Section 1 for Positive Moment *		Design Section 1 for Negative Moment *	Design Section 2 for Positive Moment **		Design Section 2 for Negative Moment **	Design Section 3 for Negative Moment ***
	n	3n		n	3n		
$A_c$ (in <sup>2</sup> )	228.38	136.96	102.98	254.38	162.96	128.98	191.98
$I_c$ (in <sup>4</sup> )	189,316	139,281	98,220	235,171	176,064	132,855	223,679
$Y_{slabc}$ (in)	22.67	34.68	44.58	24.91	36.26	44.58	47.53
$Y_{tc}$ (in)	11.92	23.93	33.83	14.16	25.51	33.83	36.78
$Y_{bc}$ (in)	60.33	48.32	38.42	59.34	47.99	39.67	39.72
$S_{tc}$ (in <sup>3</sup> )	15,882	5820	2903	16,608	6902	3927	6082
$S_{bc}$ (in <sup>3</sup> )	3138	2882	2556	3963	3669	3349	5631

\* Design Section 1 is from abutment to field splice  
 \*\* Design Section 2 is from field splice to flange butt splice  
 \*\*\* Design Section 3 is section over pier

**C. Select Applicable  
Load Combinations  
and Load Factors  
[1.3.3-1.3.5]  
[3.4.1]**

The following load multipliers will be used for this example.

$$\eta_D = 1.00$$

$$\eta_R = 1.00$$

$$\eta_I = 1.00$$

Standard HL-93 loading will be used. The load combinations considered applicable to the design example are identified below:

STRENGTH I:  $1.25 \cdot DC + 1.75 \cdot LL$

Primary applications include:

- maximum bottom flange stress in positive moment location
- maximum top and bottom flange stress in negative moment locations

STRENGTH IV:  $1.5 \cdot DC$

Primary applications include:

- maximum bottom flange stress in positive moment location
- maximum top and bottom flange stress in negative moment locations

SERVICE II:  $1.0 \cdot DC + 1.3 \cdot LL$

Corresponds to the overload provisions in the AASHTO Standard Specifications pertaining to yield control and to slip-critical connections.

FATIGUE:  $0.75 \cdot LL_{range}$

Checks to limit the potential for fatigue cracks to form in a structure.

**[3.4.2]**

CONSTRUCTION LOAD COMBINATION:  $1.25 \cdot DC_{temp} + 1.5 \cdot LL_{temp}$

During erection, the girder will need to resist stresses associated with the steel section alone. In addition, the need for diaphragms or cross frames will be determined at this stage.

Due to the continuous configuration, maximum and minimum ( $\gamma_p$ ) load factor values will be used.

IM = 15% when evaluating fatigue and fracture

IM = 33% when evaluating all other limit states

**D. Live Load  
Distribution Factors  
(LLDF) for Moment  
[4.6.2.2.2b]  
[C4.6.2.2.1-1]**

**1. Interior Beam Moment LLDFs**

For LRFD Table 4.6.2.2.1-1, a Type (a) superstructure describes the structural system used in this example. Per LRFD Table 4.6.2.2.2b-1, the approximate distribution equations can be used if these geometric constraints are met:

**Type (a) Cross Section Range of Applicability Limits for Flexure**

Parameter	Design Example	Minimum	Maximum
Beam Spacing (S)	11.33'	3.5'	16.0'
Slab Thickness ( $t_s$ )	9.0"	4.5"	12"
Number of Beams ( $N_b$ )	5	4	-
Span Length (L)	175'	20'	240'

**[4.6.2.2.1]**

In addition to S,  $t_s$ , and L, the distribution equations for live load moment area also based on  $K_g$ , a longitudinal stiffness parameter defined as:

$$K_g = n \cdot (I + A \cdot e_g^2)$$

where n is the modular ratio, I is the non-composite girder moment of inertia, A is the non-composite area of the girder, and  $e_g$  is the distance between the centers of gravity of the non-composite girder and the deck.

**Positive Moment Region**

For the positive moment region,

$$I = 77,179 \text{ in}^4$$

$$A = 91.25 \text{ in}^2$$

$$t_s = 9.0 \text{ in}$$

$$y_t = 38.96 \text{ in}$$

$$e_g = \text{concrete stool} + \frac{t_s}{2} + y_t = 1.75 + \frac{9.0}{2} + 38.96 = 45.21 \text{ in}$$

$$K_g = 8 \cdot (77,179 + 91.25 \cdot 45.21^2) = 2.11 \times 10^6 \text{ in}^4$$

Moment LLDF for one design lane loaded:

**[Table  
4.6.2.2.2b-1]**

$$\begin{aligned} gM &= 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.06 + \left(\frac{11.33}{14}\right)^{0.4} \left(\frac{11.33}{175}\right)^{0.3} \left(\frac{2.11 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.477 \text{ lanes/girder} \end{aligned}$$

Moment LLDF for two or more design lanes loaded:

$$\begin{aligned} gM &= 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.075 + \left(\frac{11.33}{9.5}\right)^{0.6} \left(\frac{11.33}{175}\right)^{0.2} \left(\frac{2.11 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.739 \text{ lanes/girder} \end{aligned} \quad \text{GOVERNS}$$

Moment LLDF for fatigue:

**[3.6.1.1.2]  
[3.6.1.4]**

The design fatigue truck is a single lane vehicle that does not include the multiple presence factor. The tabulated approximate distribution factor equations for moment include the multiple presence factors. Therefore, remove the single lane multiple presence factor (1.2) from the LLDF for one lane loaded to get the fatigue LLDF.

$$gM_f = \frac{0.477}{1.2} = 0.398 \text{ lanes/girder}$$

### Negative Moment Region

For the negative moment region (defined as between the dead load contraflexure points) use a single LLDF based on the largest negative moment section (located over the pier).

$$I = 200,770 \text{ in}^4$$

$$A = 180.25 \text{ in}^2$$

$$t_s = 9.0 \text{ in}$$

$$y_t = 39.57 \text{ in}$$

$$e_g = \text{concrete stool} + \frac{t_s}{2} + y_t = 1.75 + \frac{9.0}{2} + 39.57 = 45.82 \text{ in}$$

$$K_g = 8 \cdot (200,770 + 180.25 \cdot 45.82^2) = 4.634 \times 10^6 \text{ in}^4$$

Moment LLDF for one design lane loaded:

[Table  
4.6.2.2.2b-1]

$$\begin{aligned} gM &= 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.06 + \left(\frac{11.33}{14}\right)^{0.4} \left(\frac{11.33}{175}\right)^{0.3} \left(\frac{4.634 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.512 \text{ lanes/girder} \end{aligned}$$

Moment LLDF for two or more design lanes loaded:

[Table  
4.6.2.2.2b-1]

$$\begin{aligned} gM &= 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.075 + \left(\frac{11.33}{9.5}\right)^{0.6} \left(\frac{11.33}{175}\right)^{0.2} \left(\frac{4.634 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.793 \text{ lanes/girder} \end{aligned} \quad \underline{\text{GOVERNS}}$$

Moment LLDF for fatigue:

$$gM_f = \frac{0.512}{1.2} = 0.426 \text{ lanes/girder}$$

[4.6.2.2.2d]

## 2. Exterior Beam Moment LLDFs

Table 4.6.2.2.2d-1 contains the approximate distribution factor equations for exterior beams. Check the value of  $d_e$  to ensure they are valid.

$$d_e = 3.00 - 1.67 = 1.33 \text{ ft} \quad (\text{see Figure 6.9.5})$$

$$-1.0 \text{ ft} < 1.33 \text{ ft} < 5.5 \text{ ft} \quad \underline{\text{OK}}$$

Moment LLDF for one design lane loaded:  
 Use the lever rule and refer to Figure 6.9.5.

Exterior beam reaction or distribution factor is:

$$W_1 = W_2 = 0.5 \text{ lanes}$$

$$gM = \left[ \frac{0.5 \cdot (11.33 - 0.67) + 0.5 \cdot (11.33 - 6.67)}{11.33} \right] \cdot 1.2$$

$$= 0.811 \text{ lanes/girder}$$

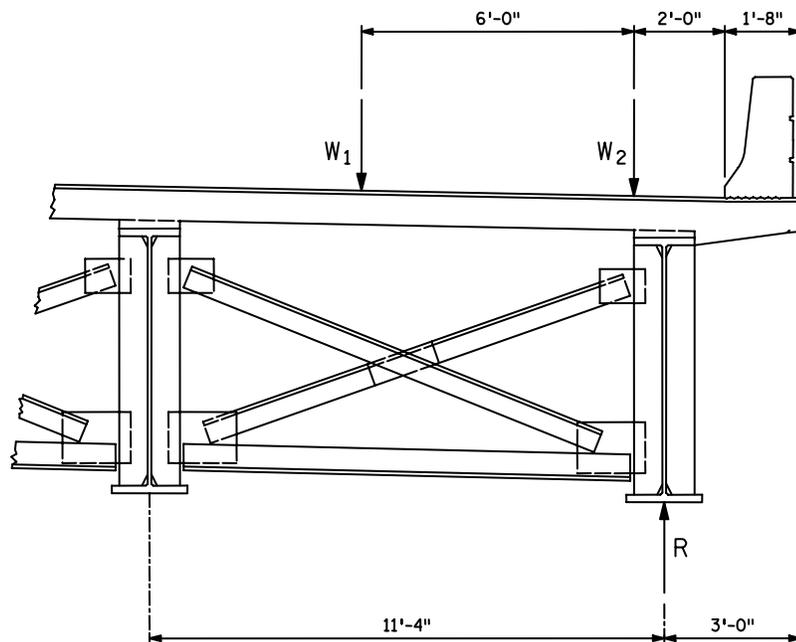


Figure 6.9.5

Moment LLDF for two or more design lanes loaded:

[Table  
 4.6.2.2.2d-1]

$$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{1.33}{9.1} = 0.916$$

$$gM = e \cdot gM_{int} = 0.916 \cdot 0.739 = 0.677 \text{ lanes/girder for pos. moment}$$

$$gM = e \cdot gM_{int} = 0.916 \cdot 0.793 = 0.726 \text{ lanes/girder for neg. moment}$$

Moment LLDFs for fatigue:

$$gM_f = \frac{0.811}{1.2} = 0.676 \text{ lanes/girder}$$

### 3. Skew Reduction Factor for Moment

[4.6.2.2.2e]

The framing plan is skewed 20 degrees. Although there is no modification to the moments for skew until the skew angle is 30 degrees or greater, note that Mn/DOT has set this factor to 1.0 for all steel bridges in order to boost the load rating.

#### E. Live Load Distribution Factors (LLDF) for Shear

### 1. Interior Beam Shear LLDFs

Check range of applicability for use of the simplified distribution equations.

[Table  
4.6.2.2.3a-1]

#### Type (a) Cross Section Range of Applicability Limits for Shear

Parameter	Design Example	Minimum	Maximum
Beam Spacing (S)	11.33'	3.5'	16.0'
Slab Thickness ( $t_s$ )	9.0"	4.5"	12"
Number of Beams ( $N_b$ )	5	4	-
Span Length (L)	175'	20'	240'
Pos. Mom. Long. Stiffness ( $K_g$ )	$2.110 \times 10^6$	10,000	$7.0 \times 10^6$
Neg. Mom. Long. Stiffness ( $K_g$ )	$4.634 \times 10^6$	10,000	$7.0 \times 10^6$

[4.6.2.2.3a]

All parameters for the design example are within permissible limits.

[Table  
4.6.2.2.3a-1]

Shear LLDF for one design lane loaded:

$$gV = 0.36 + \frac{S}{25} = 0.36 + \frac{11.33}{25} = 0.813 \text{ lanes/girder}$$

Shear LLDF for two or more design lanes loaded:

$$gV = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 = 0.2 + \frac{11.33}{12} - \left(\frac{11.33}{35}\right)^2 = 1.039 \text{ lanes/girder}$$

[4.6.2.2.3b]

### 2. Exterior Beam Shear LLDFs

Shear LLDF for one design lane loaded:

Use the lever rule, which results in the same factor that was computed for flexure.

$$gV = 0.811 \text{ lanes/girder}$$

Shear LLDF for two or more design lanes loaded:

$$e = 0.6 + \frac{d_e}{10} = 0.6 + \frac{1.33}{10} = 0.733$$

$$gV = e \cdot gV_{int} = 0.733 \cdot 1.039 = 0.762 \text{ lanes/girder}$$

**[4.6.2.2.3c]**

**3. Skew Correction Factor for Shear**

There is a modification to the shear at the obtuse corners for bridges with skewed lines of support. This example has a skew angle of 20 degrees.

**[Table  
4.6.2.2.3c-1]**

**Type (a) Cross Sections Range of Applicability Limits for Skew Correction (Shear)**

Parameter	Design Example	Minimum	Maximum
Skew Angle ( $\theta$ )	20 degrees	0 degrees	60 degrees
Beam Spacing (S)	11.33'	3.5'	16.0'
Number of Beams ( $N_b$ )	5	4	-
Span Length (L)	175'	20'	240'

$$CF = 1.0 + 0.2 \cdot \left( \frac{12 \cdot L \cdot (t_s)^3}{K_g} \right)^{0.3} \cdot \tan(\theta)$$

$$= 1.0 + 0.2 \cdot \left( \frac{12 \cdot 175 \cdot (9)^3}{2.110 \cdot 10^6} \right)^{0.3} \cdot \tan(20)$$

$$= 1.07 \text{ lanes/girder at the abutment}$$

$$CF = 1.0 + 0.2 \cdot \left( \frac{12 \cdot 175 \cdot (9)^3}{4.634 \cdot 10^6} \right)^{0.3} \cdot \tan(20)$$

$$= 1.05 \text{ lanes/girder at the pier}$$

For simplicity, only the larger correction factor will be used to modify the live load distribution factors for shear. The adjusted shear distribution factors are:

Interior Girders

For one lane loaded:

$$gV = 0.813 \cdot 1.07 = 0.870 \text{ lanes/girder}$$

For two or more design lanes loaded:

$$gV = 1.039 \cdot 1.07 = 1.112 \text{ lanes/girder}$$

For fatigue:

$$gV_f = \frac{0.870}{1.2} = 0.725 \text{ lanes/girder}$$

Exterior Girders

For one lane loaded:

$$gV = 0.811 \cdot 1.07 = 0.868 \text{ lanes/girder}$$

For two or more design lanes loaded:

$$gV = 0.762 \cdot 1.07 = 0.815 \text{ lanes/girder}$$

For fatigue:

$$gV_f = \frac{0.868}{1.2} = 0.723 \text{ lanes/girder}$$

**Table 6.9.3  
Distribution Factor Summary (Lanes/Girder)**

Girder/Force Component		One Lane LLDF	Multiple Lane LLDF	Governing LLDF
Interior Girder	+ Moment	0.477	0.739	0.739
	- Moment	0.512	0.793	0.793
	Shear	0.870	1.112	1.112
	+ Fatigue Moment	0.398		0.398
	- Fatigue Moment	0.426		0.426
	Fatigue Shear	0.725		0.725
Exterior Girder	+ Moment	0.811	0.677	0.811
	- Moment	0.811	0.726	0.811
	Shear	0.868	0.815	0.868
	+ Fatigue Moment	0.676		0.676
	- Fatigue Moment	0.676		0.676
	Fatigue Shear	0.723		0.723

**F. Calculate Force Effects**

Axial loads generated as a result of creep, shrinkage, and thermal movements will not be considered for the design of the girders. These loads are considered in the bearing and substructure design examples.

From this point forward only the design of an interior girder subject to dead load and HL-93 live loads will be presented.

**[6.10.1.5]**

Unfactored bending moments, shears, and reactions at different locations along the girder are presented in Tables 6.9.5 through 6.9.12. They are based on applying the loads as follows:

- DC1 loads are applied to a continuous beam model with varying non-composite section properties (see Table 6.9.1).
- DC2 loads are applied to a composite continuous beam model consisting of the steel girder plus the concrete deck where a modular ratio of 3n is used for the section properties.
- Live loads are applied to a composite continuous beam model consisting of the steel girder plus the concrete deck with a modular ratio of n.

Table 6.9.4 presents the areas and moments of inertia used for analysis.

**Table 6.9.4  
Composite Section Properties for Analysis**

Parameter	Design Section 1 *		Design Section 2 **		Design Section 3 ***	
	n	3n	n	3n	n	3n
A <sub>c</sub> (in <sup>2</sup> )	228.38	136.96	254.38	162.96	317.38	225.96
I <sub>c</sub> (in <sup>4</sup> )	189,316	139,281	235,171	176,064	364,078	277,092

\* Design Section 1 is from abutment to field splice

\*\* Design Section 2 is from field splice to flange butt splice

\*\*\* Design Section 3 is section over pier

DC1 consists of the following loads: girder selfweight, concrete deck and wearing course, stool, and form loads. Note that Mn/DOT includes the wearing course load with DC1 loads (not DW). A 15% detail factor (based on the selfweight of the girder) is used to account for the dead load of connection and cross frame elements. A 0.010 ksf load is considered during construction to account for the weight of deck formwork.

$$w_{\text{beam}} = \frac{A_{\text{beam}}}{144} \cdot 0.490 \cdot (1.15) = \begin{pmatrix} 0.357 \text{ k/ft - Section 1} \\ 0.459 \text{ k/ft - Section 2} \\ 0.705 \text{ k/ft - Section 3} \end{pmatrix}$$

$$\begin{aligned}
 w_{\text{deck}} &= (\text{Area Deck} + \text{Area Stool}) \cdot 0.150 \\
 &= \left( 11.33 \cdot \frac{9.5}{12} + \frac{5.25 - \text{top flange}}{12} \cdot \frac{20}{12} \right) \cdot 0.15 \\
 &= \begin{pmatrix} 1.434 \text{ k / ft} - \text{Section 1} \\ 1.418 \text{ k / ft} - \text{Section 2} \\ 1.387 \text{ k / ft} - \text{Section 3} \end{pmatrix}
 \end{aligned}$$

$$w_{\text{forms}} = 0.010 \cdot 11.33 = 0.113 \text{ k/ft}$$

A 0.020 ksf temporary construction live loading is also considered. It is assumed to be acting full length on a single span concurrent with wet concrete placement. In Table 6.9.6,  $DC_{\text{const}}$  consists of girder selfweight, form load, and one span of concrete.  $LL_{\text{const}}$  consists of one span of construction liveload.

DC2 consists of long-term dead loads, barrier, and future wearing course (FWC). Note that Mn/DOT uses a FWC of 0.020 ksf and includes the FWC load with DC2 loads (not DW).

$$w_{\text{barrier}} = 0.439 \text{ k/ft} \cdot \frac{2 \text{ barriers}}{5 \text{ girders}} = 0.176 \text{ k/ft}$$

$$w_{\text{fwc}} = 0.020 \text{ ksf} \cdot \frac{48}{5} = 0.192 \text{ k/ft}$$

The field splice is located 121 feet from the abutment bearing, approximately 0.69 of the span. This location was chosen as the nearest even foot along the span to the noncomposite dead load inflection point during the initial sizing.

All of the DC1 loads presented in the example include the 0.010 ksf load associated with the formwork. This increases the strength design loads by 2% but greatly simplifies the calculations. In reality, the load is applied to the non-composite section but is removed from the composite section. The actual stresses are also dependent on the pour sequence for the deck.

In the following tables, Girder Point 0.0 is the centerline of bearing at the abutment. Girder Point 1.0 is centerline of bearing at the pier. Due to the symmetry of the span arrangement, only data for Girder Points 0.0 to 1.0 is provided for most loads. However, due to the asymmetric loading

considered during construction, values are provided for both spans in Tables 6.9.6 and 6.9.13.

**Table 6.9.5**  
**Dead Load Bending Moments (Unfactored)**

Girder Point	DC1 Moment (k-ft)				DC2 Moment (k-ft)		
	Girder	Slab and Stool	Forms	Total	Barrier	FWC	Total
0.0	0	0	0	0	0	0	0
0.1	329	1297	102	1728	163	178	341
0.2	549	2154	170	2873	273	297	570
0.3	659	2573	202	3434	328	357	685
0.4	659	2552	202	3413	329	359	688
0.5	550	2092	165	2807	277	302	579
0.6	331	1194	94	1619	170	186	356
0.691 <sup>(1)</sup>	32	-13	-1	18	26	28	54
0.7	-2	-145	-11	-158	10	11	21
0.742 <sup>(2)</sup>	-186	-853	-67	-1106	-75	-82	-157
0.8	-470	-1922	-152	-2544	-204	-222	-426
0.807 <sup>(3)</sup>	-510	-2066	-163	-2738	-221	-242	-463
0.860 <sup>(4)</sup>	-819	-3197	-252	-4268	-358	-391	-749
0.871 <sup>(5)</sup>	-894	-3458	-272	-4624	-390	-425	-815
0.9	-1092	-4134	-326	-5552	-472	-515	-987
0.936 <sup>(6)</sup>	-1364	-5029	-396	-6790	-581	-633	-1214
1.0	-1918	-6778	-535	-9231	-794	-866	-1660

<sup>(1)</sup> Field splice

<sup>(2)</sup> Second diaphragm away from pier

<sup>(3)</sup> Midway point between first and second diaphragms away from pier

<sup>(4)</sup> Flange butt splice

<sup>(5)</sup> First diaphragm away from pier

<sup>(6)</sup> Midway point between centerline of pier and first diaphragm away from pier

For this design example, the LRFD 6.10.3.2 constructibility checks use the values provided in Table 6.9.6.

**Table 6.9.6**  
**Construction Load Bending Moments (Unfactored)**

Girder Point	DC <sub>const</sub> Moment (kip-ft)			LL <sub>const</sub> Moment (kip-ft)	
	Girder	Slab and Stool	Forms		Total
0.0	0	0	0	0	0
0.1	329	1677	102	2108	259
0.2	549	2902	170	3621	448
0.297 <sup>(1)</sup>	657	3663	202	4522	566
0.3	659	3679	202	4540	568
0.365 <sup>(2)</sup>	670	3957	205	4832	612
0.4	659	4003	202	4864	618
0.446 <sup>(3)</sup>	623	4003	189	4815	618
0.5	550	3879	165	4594	599
0.6	331	3305	94	3730	511
0.691 <sup>(4)</sup>	32	2385	-1	2416	369
0.7	-2	2279	-11	2266	353
0.8	-470	808	-152	186	126
0.860 <sup>(5)</sup>	-819	-289	-252	-1360	-43
0.9	-1092	-1109	-326	-2527	-170
1.0	-1918	-3471	-535	-5924	-538
1.1	-1092	-3124	-326	-4542	-483
1.2	-470	-2777	-152	-3399	-429
1.3	-2	-2430	-11	-2443	-376
1.309 <sup>(4)</sup>	32	-2400	-1	-2369	-371
1.4	331	-2082	94	-1657	-322
1.5	550	-1736	165	-1021	-268
1.6	659	-1390	202	-529	-215
1.7	659	-1041	202	-180	-162
1.8	549	-695	170	24	-108
1.9	329	-347	102	84	-54
2.0	0	0	0	0	0

<sup>(1)</sup> Second diaphragm away from abutment

<sup>(2)</sup> Midway point between second and third diaphragm away from abutment

<sup>(3)</sup> Third diaphragm away from abutment

<sup>(4)</sup> Field splice

<sup>(5)</sup> Flange butt splice

The truck train generated the controlling negative bending moment over the pier. The distance between trucks in the train is variable but can be no less than 50 feet. The largest moment was obtained when the distance between the last axle of the first truck and the first axle of the second truck was 119 feet. The truck train multiplier was increased from 0.90 to 1.05 based on the Memo to Designers (2005-01).

Table 6.9.7 contains positive and negative live load moments due to truck, lane, and truck train loading.

**Table 6.9.7**  
**Live Load Design Moments per Lane (Unfactored)**

Girder Point	Pos. M.* (kip-ft)	Neg. M.** (kip-ft)	Governing LL Type for Negative Moment
0.0	0	0	(Truck + IM) + Lane
0.1	2064	-329	(Truck + IM) + Lane
0.2	3510	-658	(Truck + IM) + Lane
0.3	4428	-987	(Truck + IM) + Lane
0.4	4783	-1316	(Truck + IM) + Lane
0.5	4648	-1645	(Truck + IM) + Lane
0.6	4073	-1973	(Truck + IM) + Lane
0.691 <sup>(1)</sup>	3182	-3113	(Truck Train + IM) + Lane
0.7	3085	-3153	(Truck Train + IM) + Lane
0.742 <sup>(2)</sup>	2565	-3358	(Truck Train + IM) + Lane
0.8	1784	-3642	(Truck Train + IM) + Lane
0.807 <sup>(3)</sup>	1696	-3694	(Truck Train + IM) + Lane
0.860 <sup>(4)</sup>	1025	-4041	(Truck Train + IM) + Lane
0.871 <sup>(5)</sup>	902	-4174	(Truck Train + IM) + Lane
0.9	625	-4584	(Truck Train + IM) + Lane
0.936 <sup>(6)</sup>	341	-5254	(Truck Train + IM) + Lane
1.0	0	-6905	(Truck Train + IM) + Lane

\* Positive M = (1.33 x Truck) + Lane

\*\* Negative M = maximum of (1.33 x Truck) + Lane or 1.05 x [(1.33 x Truck Train) + Lane]

<sup>(1)</sup> Field splice

<sup>(2)</sup> Second diaphragm away from pier

<sup>(3)</sup> Midway point between first and second diaphragms away from pier

<sup>(4)</sup> Flange butt splice

<sup>(5)</sup> First diaphragm away from pier

<sup>(6)</sup> Midway point between centerline of pier and first diaphragm away from pier

Table 6.9.8 lists the fatigue moment range at various girder points when the fixed axle fatigue truck is run across the structural model.

**Table 6.9.8**  
**Live Load Fatigue Moments per Lane (Unfactored)**

Girder Point	Fatigue Moment Range (kip-ft) *
0.0	0
0.1	1248
0.2	2136
0.3	2691
0.4	2978
0.5	3053
0.6	2959
0.691 <sup>(1)</sup>	2691
0.7	2659
0.8	2209
0.860 <sup>(2)</sup>	1908
0.9	1759
1.0	1567

\* Fatigue Moment Range = 1.15 · (Fatigue Truck Positive M - Fatigue Truck Neg. M)

<sup>(1)</sup> Field Splice

<sup>(2)</sup> Flange Butt Splice

Table 6.9.9 presents the unfactored dead load shear forces at different girder locations for different load components.

**Table 6.9.9  
Dead Load Shear (Unfactored)**

Girder Point	DC1 Shear (kips)				DC2 Shear (kips)		
	Girder	Slab and Stool	Forms	Total	Barrier	FWC	Total
0.0	22	87	7	115	11	12	23
0.1	16	62	5	82	8	8	16
0.2	9	36	3	49	5	5	10
0.3	3	11	1	15	1	2	3
0.4	-3	-14	-1	-18	-1	-2	-3
0.5	-9	-39	-3	-51	-4	-5	-9
0.6	-16	-64	-5	-85	-8	-8	-16
0.691 <sup>(1)</sup>	-22	-87	-7	-116	-11	-11	-22
0.7	-23	-89	-7	-119	-11	-11	-22
0.742 <sup>(2)</sup>	-26	-100	-8	-134	-12	-13	-25
0.8	-31	-114	-9	-154	-14	-15	-29
0.860 <sup>(3)</sup>	-37	-129	-10	-176	-16	-17	-33
0.871 <sup>(4)</sup>	-38	-132	-10	-180	-16	-17	-33
0.9	-41	-139	-11	-191	-17	-18	-35
1.0	-53	-163	-13	-230	-20	-22	-42

- <sup>(1)</sup> Field splice
- <sup>(2)</sup> Second diaphragm away from pier
- <sup>(3)</sup> Flange butt splice
- <sup>(4)</sup> First diaphragm away from pier

Table 6.9.10 contains the dead load reactions at Abutment, (Girder Point 0.0) and Pier (Girder Point 1.0). The reactions at Girder Point 1.0 are larger than the shear at Girder Point 0.0 because the reaction includes the load from both spans.

**Table 6.9.10  
Dead Load Reactions (Unfactored)**

Girder Point	DC1 Reaction (kips)	DC2 Reaction (kips)
0.0	115	23
1.0	459	83

Table 6.9.11 contains the live load shear extremes for the various live load components. Per LRFD Article 3.6.1.3.1, truck train loading is not to be used for shear.

**Table 6.9.11**  
**Live Load Design Shear per Lane and Fatigue Shear (Unfactored)**

Girder Point	Truck + Lane Positive Shear* (kips)	Truck + Lane Negative Shear* (kips)	Fatigue Truck Shear Range** (kips)
0.0	137	-19	78
0.1	116	-20	67
0.2	95	-31	57
0.3	75	-47	56
0.4	58	-63	58
0.5	43	-80	60
0.6	30	-98	62
0.691 <sup>(1)</sup>	20	-114	64
0.7	19	-115	65
0.742 <sup>(2)</sup>	15	-122	66
0.8	10	-132	69
0.860 <sup>(3)</sup>	6	-142	71
0.871 <sup>(4)</sup>	5	-144	71
0.9	4	-149	72
1.0	0	-166	76

\* = (1.33 · (Truck Shear)) + Lane Shear

\*\* = 1.15 · (Fatigue Truck Positive V - Fatigue Truck Negative V)

<sup>(1)</sup> Field splice

<sup>(2)</sup> Second diaphragm away from pier

<sup>(3)</sup> Flange butt splice

<sup>(4)</sup> First diaphragm away from pier

Table 6.9.12 presents the live load reactions at the abutment (Girder Point 0.0) and the pier (Girder Point 1.0). Similar to the dead load reactions presented in Table 6.9.10, the reactions at Girder Point 1.0 are larger than the shear at Girder Point 0.0 because the reaction includes the load from both spans. Per LRFD Article 3.6.1.3.1 the truck train loading needs to be considered for reactions at interior supports.

**Table 6.9.12**  
**Live Load Reactions per Lane (Unfactored)**

Girder Point	LL + IM Reaction (kips)	LL Only Reaction (kips)
0.0	137	115
1.0	341*	294

\* HL-93 Truck Train + Lane Reaction governs

Table 6.9.13 presents shear values due to construction loads.

**Table 6.9.13**  
**Construction Load Shear (Unfactored)**

Girder Point	DC <sub>const</sub> Shear (kips)				LL <sub>const</sub> Shear (kips)
	Girder	Slab+Stool	Forms	Total	
0.0	22	108	7	137	18
0.1	16	82	5	103	13
0.2	9	57	3	69	9
0.3	3	32	1	36	4
0.4	-3	6	-1	2	1
0.5	-9	-20	-3	-32	-3
0.6	-16	-45	-5	-66	-8
0.691 <sup>(1)</sup>	-22	-69	-7	-98	-11
0.7	-23	-71	-7	-101	-11
0.8	-31	-97	-9	-137	-15
0.860 <sup>(2)</sup>	-37	-112	-10	-159	-18
0.9	-41	-122	-11	-174	-19
1.0 Left	-53	-148	-13	-214	-23
1.0 Right	53	20	13	86	3
1.1	41	20	11	72	3
1.140 <sup>(2)</sup>	37	20	10	67	3
1.2	31	20	9	60	3
1.3	23	20	7	50	3
1.309 <sup>(1)</sup>	22	20	7	49	3
1.4	16	19	5	40	3
1.5	9	20	3	32	3
1.6	3	20	1	24	3
1.7	-3	20	-1	16	3
1.8	-9	20	-3	8	3
1.9	-16	20	-5	-1	3
2.0	-22	20	-7	-9	3

<sup>(1)</sup> Field splice

<sup>(2)</sup> Flange butt splice

The checks in this example begin with the strength checks on the preliminary layout. Designers should be aware that deflections may control the design. The deflection checks for this example are presented in Section M.

**G. Flexure –  
Investigate  
Strength Limit  
State**

At the strength limit state the girder is designed to carry factored dead and live loads. The resisting section in the positive moment regions is the girder plus deck composite section. In the negative moment regions, resistance is provided by the girder plus deck reinforcement composite section.

**G.1 Design  
Section 1 –  
Positive Moment**

The maximum factored positive moment  $M_u$  is at  $0.4L = 70.0$  ft from each abutment.

$$\begin{aligned} M_u &= 1.25 \cdot (3413 + 688) + 1.75 \cdot (4783) \cdot 0.739 \\ &= 5126 + 6186 = 11312 \text{ kip-ft} \end{aligned}$$

The maximum factored stresses are at  $0.4L$  for top and bottom flanges. Refer back to Tables 6.9.1 and 6.9.2 for section properties and Tables 6.9.5 & 6.9.7 for moments.

For top flange:

$$\begin{aligned} f_{\text{buc}} &= 1.25 \left( \frac{M_{\text{DC1}}}{S_t} + \frac{M_{\text{DC2}}}{S_{\text{tc}(3n)}} \right) + 1.75 \left( \frac{M_{\text{LL+I}}}{S_{\text{tc}(n)}} \right) (\text{LLDF}) \\ &= 1.25 \left( \frac{3413}{1981} + \frac{688}{5820} \right) (12) + 1.75 \left( \frac{4783}{15882} \right) (12) (0.739) \\ &= 32.3 \text{ ksi} \end{aligned}$$

For bottom flange:

$$\begin{aligned} f_{\text{but}} &= 1.25 \left( \frac{M_{\text{DC1}}}{S_b} + \frac{M_{\text{DC2}}}{S_{\text{bc}(3n)}} \right) + 1.75 \left( \frac{M_{\text{LL+I}}}{S_{\text{bc}(n)}} \right) (\text{LLDF}) \\ &= 1.25 \left( \frac{3413}{2318} + \frac{688}{2882} \right) (12) + (1.75) \left( \frac{4783}{3138} \right) (12) (0.739) \\ &= 49.3 \text{ ksi} \end{aligned}$$

Since the bridge only has a minor skew, and the beam is an interior beam, lateral bending does not need to be considered.

$$f_\ell = 0.0$$

The procedure for evaluating the flexural strength of a girder in accordance with the LRFD Specifications is quite involved. To clarify the steps involved, flow charts are included in LRFD Appendix C, Article C6.4. Follow the procedure shown in LRFD Figures C6.4.4-1 and C6.4.5-1.

The span under consideration is continuous, but Mn/DOT does not allow design using the moment redistribution provisions of LRFD Appendix B. In addition, Mn/DOT does not permit exceeding the moment at first yield for all sections at positive moment region. Therefore, there is no need to check section compactness criteria and the check will be made in accordance with LRFD 6.10.7.2 (see LRFD Figure C6.4.5-1).

### Compression Flange in Positive Flexure

[6.10.7.2.2]

The nominal flexural resistance of the compression flange shall be taken as:

$$F_{nc} = R_b R_h F_{yc}$$

[6.10.1.10.2]

First determine  $R_b$  :

[6.10.2.1.1]

$$\frac{D}{t_w} = \frac{70}{0.625} = 112 < 150 \quad \underline{OK}$$

The section is composite in positive flexure. Therefore web load-shedding factor  $R_b = 1.0$ .

[6.10.1.10.1]

The section is homogenous. Therefore hybrid factor  $R_h = 1.0$ .

$$F_{yc} = 50 \text{ ksi}$$

$$\text{Then, } F_{nc} = (1.0) (1.0) (50.0) = 50 \text{ ksi}$$

Compression flange shall satisfy:

[6.10.7.2.1]

$$f_{buc} \leq \phi_f F_{nc} \quad \phi_f = 1.0$$

$$f_{buc} = 32.3 \text{ ksi} < (1.0) (50.0) = 50.0 \text{ ksi} \quad \underline{OK}$$

**Tension Flange in Positive Flexure**

[6.10.7.2.2]

The nominal flexure resistance of tension flange shall be taken as:

$$F_{nt} = R_h F_{yt}$$

The section is homogenous, so  $R_h = 1.0$

$$F_{yt} = 50.0 \text{ ksi}$$

$$F_{nt} = (1.0) (50.0) = 50.0 \text{ ksi}$$

Tension flange shall satisfy:

[6.10.7.2.1]

$$f_{but} + \frac{1}{3} f_{\ell} \leq \phi_f F_{nt}$$

$$f_{but} = 49.3 \text{ ksi} \quad f_{\ell} = 0.0 \text{ ksi} \quad \phi_f = 1.0$$

$$f_{but} + \frac{1}{3} f_{\ell} = 49.3 + 0.0 = 49.3 \text{ ksi} < (1.0) (50.0) = 50.0 \text{ ksi} \quad \text{OK}$$

The positive moment section has adequate flexural strength.

[6.10.7.3]

Check ductility of the section:

[D6.1]

To determine  $D_p$  use Appendix D from Section 6. The figure for load components for positive bending sections is presented in Figure 6.9.6.

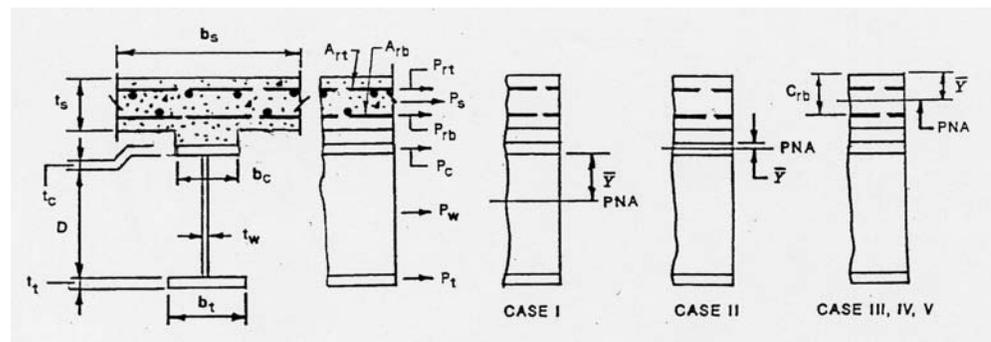


Figure 6.9.6

To simplify computations neglect the  $P_{rt}$  and  $P_{rb}$  terms.

$$P_c = \text{Force in the top flange} = 50 \cdot 1.0 \cdot 20 = 1000 \text{ kips}$$

$$P_w = \text{Force in the web} = 50 \cdot 0.625 \cdot 70 = 2188 \text{ kips}$$

$$P_t = \text{Force in the bottom flange} = 50 \cdot 1.25 \cdot 22 = 1375 \text{ kips}$$

$$P_s = \text{Force in the slab} = 0.85 \cdot 4 \cdot (9 \cdot 118 + 1.75 \cdot 20) = 3730 \text{ kips}$$

Begin by checking Case I (PNA in the web of the girder).

$$P_t + P_w = 1375 + 2188 = 3563 \text{ kips}$$

$$P_c + P_s = 1000 + 3730 = 4730 > 3563$$

Therefore, the PNA is not in the web.

Try Case II (PNA in the top flange)

$$P_t + P_w + P_c = 1375 + 2188 + 1000 = 4563 \text{ kips}$$

$$P_s = 3730 < 4563 \text{ kips}$$

Therefore, the PNA is in the top flange. Use the equation in LRFD Table D6.1-1 to locate the position of the PNA in the top flange.

$$\bar{y}_{tfl} = \frac{t_c}{2} \cdot \left( \frac{P_w + P_t - P_s}{P_c} + 1 \right) = \frac{1.0}{2} \cdot \left( \frac{2188 + 1375 - 3730}{1000} + 1 \right) = 0.42 \text{ in}$$

$$D_t = t_s + t_{cstool} + d_{nc} = 9 + 1.75 + 72.25 \text{ in} = 83.0 \text{ in}$$

$$D_p = t_s + t_{cstool} + \bar{y}_{tfl} = 9 + 1.75 + 0.42 = 11.17 \text{ in}$$

$$0.42 \cdot D_t = 0.42 \cdot 83.0 = 34.86 \text{ in}$$

$$D_p = 11.17 \text{ in} < 34.86 \text{ in} \quad \underline{OK}$$

**G.2 Design  
Section 3 –  
Negative Moment**

First, determine the maximum factored stresses at pier for the top and bottom flange. Referring back to Tables 6.9.1 and 6.9.2 for section properties and Tables 6.9.5 and 6.9.7 for moments:

For top flange:

$$\begin{aligned} f_{\text{but}} &= 1.25 \cdot \left( \frac{M_{\text{DC1}}}{S_t} + \frac{M_{\text{DC2}}}{S_{\text{tc}}} \right) + 1.75 \cdot \left( \frac{M_{\text{LL+I}}}{S_{\text{tc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left( \frac{9231}{5074} + \frac{1660}{6082} \right) \cdot 12 + 1.75 \cdot \left( \frac{6905}{6082} \right) \cdot 12 \cdot 0.793 = 50.3 \text{ ksi} \end{aligned}$$

For bottom flange:

$$\begin{aligned} f_{\text{buc}} &= 1.25 \cdot \left( \frac{M_{\text{DC1}}}{S_b} + \frac{M_{\text{DC2}}}{S_{\text{bc}}} \right) + 1.75 \cdot \left( \frac{M_{\text{LL+I}}}{S_{\text{bc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left( \frac{9231}{5437} + \frac{1660}{5631} \right) \cdot 12 + 1.75 \cdot \left( \frac{6905}{5631} \right) \cdot 12 \cdot 0.793 = 50.3 \text{ ksi} \end{aligned}$$

Since bridge only has minor skew, and it is an interior beam, no lateral bending needs to be considered

$$f_\ell = 0.0 \text{ ksi}$$

Next, determine flexural resistance of top and bottom flanges. Refer to the flow chart shown in LRFD Figure C6.4.4.-1. Mn/DOT does not use the optional provisions of Appendix A, so there is no need to check the web slenderness ratio and flange inertia ratio of LRFD Article 6.10.6.2.3. Our check will be made in accordance with LRFD 6.10.8. (See LRFD Figure C6.4.6-1.) Begin with the compression (bottom) flange, which is discretely braced. The flexural resistance of the compression flange  $F_{\text{nc}}$  is the smaller of the local buckling resistance  $F_{\text{nc}(\text{FLB})}$  and the lateral torsional buckling resistance  $F_{\text{nc}(\text{LTB})}$ . First, check local buckling resistance.

**[6.10.8.2.2]**

$$\lambda_f = \frac{b_{\text{fc}}}{2t_{\text{fc}}} = \frac{22}{2 \cdot (3.25)} = 3.38$$

**[C6.10.8.2.2]**

For  $F_{\text{yc}} = 50 \text{ ksi}$ ,  $\lambda_{\text{pf}} = 9.2$

Then,  $\lambda_f < \lambda_{\text{pf}}$  and compression flange is compact.

For a compact compression flange, flexural resistance  $F_{\text{nc}}$  is

$$F_{\text{nc}(\text{FLB})} = R_b R_h F_{\text{yc}}$$

**[6.10.1.10.2]**

For a negative moment section,  $R_b$  is dependent on the web slenderness ratio  $\frac{2 \cdot D_c}{t_w}$ .

**[D6.3.1]**

For composite negative moment sections,  $D_c$  is based on the section consisting of the steel girder plus the longitudinal reinforcement.

Then,  $D_c = y_{bc} - t_{fc} = 39.72 - 3.25 = 36.47$  in

$$\frac{2 \cdot D_c}{t_w} = \frac{2 \cdot (36.47)}{0.625} = 116.7$$

**[C6.10.1.10.2]**

For  $F_y = 50$  ksi,  $\lambda_{rw} = 137$ . Therefore,

$$\frac{2 \cdot D_c}{t_w} < \lambda_{rw} \text{ and } R_b = 1.0.$$

The girder is homogeneous (not a hybrid), so  $R_h = 1.0$

The flange local buckling resistance is:

$$F_{nc(\text{FLB})} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

**[6.10.8.2.3]**

Now, determine the lateral torsional buckling resistance.

At the pier, the unbraced length  $L_b$  is:

$$L_b = 22.5 \text{ ft} = 270 \text{ in}$$

This is to be compared with the compact bracing limit  $L_p$ .

$$L_p = 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}}$$

$$r_t = \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{D_c t_w}{3 b_{fc} t_{fc}}\right)}} = \frac{22}{\sqrt{12 \cdot \left(1 + \frac{36.47 \cdot 0.625}{3 \cdot 22 \cdot 3.25}\right)}} = 6.04 \text{ in}$$

$$\text{Then, } L_p = 1.0 \cdot (6.04) \cdot \sqrt{\frac{29,000}{50}} = 145.5 \text{ in} < 270 \text{ in}$$

Therefore,  $L_b > L_p$ , so check noncompact bracing limit  $L_r$ .

$$L_r = \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}}$$

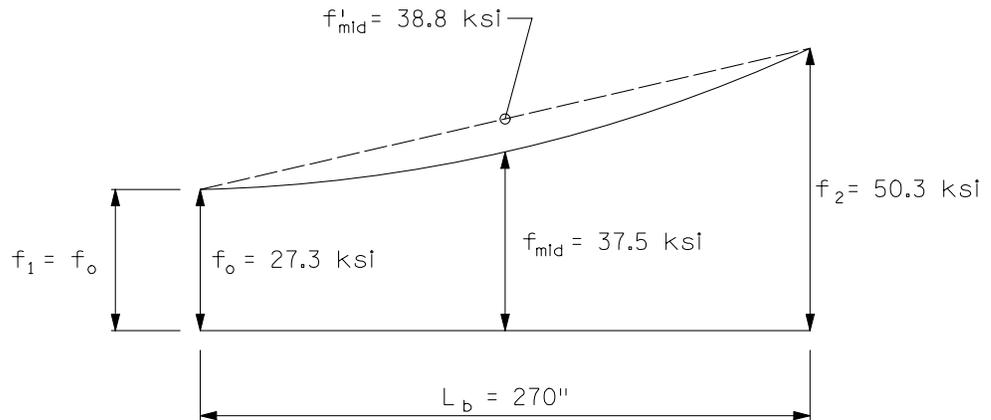
$$F_{yr} = 0.7 \cdot F_{yc} = 0.7 \cdot (50.0) = 35.0 \text{ ksi}$$

$$\text{Then, } L_r = \pi \cdot (6.04) \cdot \sqrt{\frac{29,000}{35.0}} = 546.2 \text{ in} > 270 \text{ in}$$

Therefore,  $L_p < L_b < L_r$

$$\text{And, } F_{nc(LTB)} = C_b \cdot \left[ 1 - \left( 1 - \frac{F_{yr}}{R_h \cdot F_{yc}} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$C_b$ , moment gradient modifier, can be calculated as follows (refer to Figure 6.9.7 and LRFD C6.4.10):



**Figure 6.9.7**

$f_2$  is the compression stress at centerline of pier diaphragm:

$$f_2 = 50.3 \text{ ksi}$$

$f_o$  is the compression stress at first brace point (diaphragm) away from pier:

$$\begin{aligned} f_o &= 1.25 \left( \frac{4624}{5437} + \frac{815}{5631} \right) (12) + (1.75) \left( \frac{4174}{5631} \right) (12) (0.793) \\ &= 27.3 \text{ ksi} \end{aligned}$$

$f_{mid}$  is the compression stress at the point midway between the centerline of pier and first brace point away from pier:

$$\begin{aligned} f_{mid} &= 1.25 \left( \frac{6790}{5437} + \frac{1214}{5631} \right) (12) + (1.75) \left( \frac{5254}{5631} \right) (12) (0.793) \\ &= 37.5 \text{ ksi} \end{aligned}$$

Check if moment envelope is concave between  $f_2$  and  $f_o$ . The stress at the middle of the unbraced length,  $f'_{mid}$ , assuming a linear variation between  $f_2$  and  $f_o$  is:

$$f'_{mid} = \frac{f_2 + f_o}{2} = \frac{50.3 + 27.3}{2} = 38.8 \text{ ksi} > 37.5 \text{ ksi}$$

Therefore, moment envelope is concave and  $f_1 = f_o = 27.3$  ksi

$$\frac{f_1}{f_2} = \frac{27.3}{50.3} = 0.54$$

$$\begin{aligned} C_b &= 1.75 - 1.05 \left( \frac{f_1}{f_2} \right) + 0.3 \left( \frac{f_1}{f_2} \right)^2 \\ &= 1.75 - (1.05) (0.54) + (0.3) (0.54)^2 \\ &= 1.27 < 2.3 \quad \quad \quad \underline{\text{OK}} \end{aligned}$$

Then,

$$\begin{aligned} F_{nc(LTB)} &= 1.27 \cdot \left[ 1 - \left( 1 - \frac{35}{1.0 \cdot 50} \right) \left( \frac{270 - 145.5}{546.2 - 145.5} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0 \\ &= 57.6 \text{ ksi} > 50 \text{ ksi} \end{aligned}$$

Therefore,

$$F_{nc(LTB)} = 50.0 \text{ ksi}$$

The compression flange flexural resistance  $F_{nc}$  is the smaller of  $F_{nc(FLB)}$  and  $F_{nc(LTB)}$ .

$$F_{nc} = F_{nc(LTB)} = 50.0 \text{ ksi}$$

Then,  $\phi F_{nc} = 1.0 \cdot 50.0 = 50.0 \text{ ksi}$

$$f_{\text{buc}} + \frac{1}{3} f_{\ell} = 50.3 + 0.0 = 50.3 \text{ ksi} \approx 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

**[6.10.8.1.3]**

Now, consider the tension (top) flange, which is continuously braced by the deck in its final state.

$$\text{Then, } \phi F_{\text{nt}} = \phi \cdot (R_h \cdot F_{yt}) = 1.0 \cdot (1.0 \cdot 50.0) = 50.0 \text{ ksi}$$

$$f_{\text{but}} = 50.3 \approx 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

**G.3 Design****Section 2 –****Negative Moment**

First, determine the maximum factored stresses at 0.860L (flange butt splice location) for the top and bottom flange. Referring back to Tables 6.9.1 and 6.9.2 for section properties and Tables 6.9.5 and 6.9.7 for moments:

For top flange:

$$\begin{aligned} f_{\text{but}} &= 1.25 \cdot \left( \frac{M_{\text{DC1}}}{S_t} + \frac{M_{\text{DC2}}}{S_{\text{tc}}} \right) + 1.75 \cdot \left( \frac{M_{\text{LL+I}}}{S_{\text{tc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left( \frac{4268}{2970} + \frac{749}{3927} \right) \cdot 12 + 1.75 \cdot \left( \frac{4041}{3927} \right) \cdot 12 \cdot 0.793 = 41.6 \text{ ksi} \end{aligned}$$

For bottom flange:

$$\begin{aligned} f_{\text{buc}} &= 1.25 \cdot \left( \frac{M_{\text{DC1}}}{S_b} + \frac{M_{\text{DC2}}}{S_{\text{bc}}} \right) + 1.75 \cdot \left( \frac{M_{\text{LL+I}}}{S_{\text{bc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left( \frac{4268}{3149} + \frac{749}{3349} \right) \cdot 12 + 1.75 \cdot \left( \frac{4041}{3349} \right) \cdot 12 \cdot 0.793 = 43.8 \text{ ksi} \end{aligned}$$

Since bridge only has minor skew, and it is an interior beam, no lateral bending needs to be considered:

$$f_{\ell} = 0.0 \text{ ksi}$$

Next, determine the flexural resistance of top and bottom flanges. Refer to the flow chart shown in LRFD Figure C6.4.4.-1. Mn/DOT does not use the optional provisions of Appendix A, so there is no need to check the web slenderness ratio and flange inertia ratio of LRFD Article 6.10.6.2.3. The check will be made in accordance with LRFD 6.10.8. (See LRFD Figure C6.4.6-1) Begin with the compression (bottom) flange, which is discretely braced. The flexural resistance of the compression flange  $F_{\text{nc}}$  is the smaller of the local buckling resistance  $F_{\text{nc(FLB)}}$  and the lateral torsional buckling resistance  $F_{\text{nc(LTB)}}$ .

First, check local buckling resistance.

$$[6.10.8.2.2] \quad \lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{22}{2 \cdot (1.75)} = 6.29$$

$$[C6.10.8.2.2] \quad \text{For } F_{yc} = 50 \text{ ksi, } \lambda_{pf} = 9.2$$

Then,  $\lambda_f < \lambda_{pf}$  and compression flange is compact.

For a compact compression flange, flexural resistance  $F_{nc}$  is

$$F_{nc(\text{FLB})} = R_b R_h F_{yc}$$

$$[6.10.1.10.2] \quad \text{For a negative moment section, } R_b \text{ is dependent on the web slenderness ratio, } \frac{2 \cdot D_c}{t_w}.$$

$[D6.3.1]$  For composite negative moment sections,  $D_c$  is based on the section consisting of the steel girder plus the longitudinal reinforcement.

$$D_c = y_{bc} - t_{fc} = 39.67 - 1.75 = 37.92 \text{ in}$$

$$\frac{2 \cdot D_c}{t_w} = \frac{2 \cdot (37.92)}{0.625} = 121.34$$

$[C6.10.1.10.2]$  For  $F_y = 50$  ksi,  $\lambda_{rw} = 137$ . Therefore,

$$\frac{2 \cdot D_c}{t_w} < \lambda_{rw} \text{ and } R_b = 1.0.$$

The girder is homogeneous so  $R_h = 1.0$

The flange local buckling resistance is:

$$F_{nc(\text{FLB})} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

$[6.10.8.2.3]$  Now, determine the lateral torsional buckling resistance.

At the flange butt splice, the unbraced length  $L_b$  is:

$$L_b = 22.5 \text{ ft} = 270 \text{ in}$$

This is to be compared with the compact bracing limit  $L_p$ .

[6.10.8.2.3-4]

$$L_p = 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}}$$

[6.10.8.2.3]

$$r_t = \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{D_c t_w}{3 b_{fc} t_{fc}}\right)}} = \frac{22}{\sqrt{12 \cdot \left(1 + \frac{37.92 \cdot 0.625}{3 \cdot 22 \cdot 1.75}\right)}} = 5.79 \text{ in}$$

$$\text{Then, } L_p = 1.0 \cdot (5.79) \cdot \sqrt{\frac{29,000}{50}} = 139.4 \text{ in} < 270 \text{ in}$$

Therefore,  $L_b > L_p$ , so check noncompact bracing limit  $L_r$ .

$$L_r = \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}}$$

$$F_{yr} = 0.7 \cdot F_y = 0.7 \cdot (50.0) = 35.0 \text{ ksi}$$

$$\text{Then, } L_r = \pi \cdot (5.79) \cdot \sqrt{\frac{29,000}{35.0}} = 523.6 \text{ in} > 270 \text{ in}$$

Therefore,  $L_p < L_b < L_r$

$$\text{And, } F_{nc(LTB)} = C_b \cdot \left[ 1 - \left( 1 - \frac{F_{yr}}{R_h \cdot F_{yc}} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$C_b$ , moment gradient modifier, can be calculated as follows:

$f_2$  is the compression stress at first brace point (diaphragm) off the pier:

$$f_2 = 27.3 \text{ ksi}$$

$f_o$  is the compression stress at second diaphragm away from pier:

$$f_o = 1.25 \left( \frac{1106}{3149} + \frac{157}{3349} \right) (12) + 1.75 \left( \frac{3358}{3349} \right) (12) (0.793)$$

$$= 22.7 \text{ ksi}$$

$f_{mid}$  is the compression stress at the point midway between the first and second diaphragm away from pier:

$$f_{mid} = 1.25 \left( \frac{2738}{3149} + \frac{463}{3349} \right) (12) + 1.75 \left( \frac{3694}{3349} \right) (12) (0.793)$$

$$= 33.5 \text{ ksi}$$

$$\frac{f_{mid}}{f_2} = \frac{33.5}{27.3} = 1.23 > 1 \quad \text{Therefore, } C_b = 1.0$$

$$F_{nc(LTB)} = 1.0 \cdot \left[ 1 - \left( 1 - \frac{35}{1.0 \cdot 50} \right) \left( \frac{270 - 139.4}{523.6 - 139.4} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0$$

$$= 44.90 \text{ ksi}$$

The compression flange flexural resistance  $F_{nc}$  is the smaller of  $F_{nc(FLB)}$  and  $F_{nc(LTB)}$ .

$$F_{nc} = F_{nc(LTB)} = 44.90 \text{ ksi}$$

Then,  $\phi F_{nc} = 1.0 \cdot 44.90 = 44.90 \text{ ksi}$

$$f_{buc} + \frac{1}{3} \cdot f_\ell = 43.8 < 44.9 \text{ ksi} \quad \underline{\text{OK}}$$

### [6.10.8.1.3]

Now consider the tension (top) flange, which is continuously braced by the deck in its final state.

Then

$$\phi F_{nt} = \phi \cdot (R_h \cdot F_{yt}) = 1.0 \cdot (1.0 \cdot 50.0) = 50.0 \text{ ksi} > 41.6 \text{ ksi} \quad \underline{\text{OK}}$$

**G.4 Design  
Section 1 - Negative  
Moment**

Design Section 1 was checked for negative moment following the same procedure used for Design Section 2 and found to be adequate.

**G.5 Design  
Section 2 - Positive  
Moment**

Design Section 2 was checked for positive moment following the same procedure used for Design Section 1 and found to be adequate.

**H. Flexure –  
Investigate  
Constructibility**

The capacity of the girders must be evaluated during construction, prior to composite action carrying the loads. For this example, the check consists of placing selfweight and formwork on both spans, while deck dead loads and a 20 psf construction live load is placed on one span.

**H.1 Design  
Section 1 -Positive  
Moment**

Load factors for this check are based on the values provided in LRFD Article 3.4.2, where 1.25 is used on dead loads and 1.5 is used on live loads. The maximum factored construction moment for Section 1 occurs at 0.4L:

$$\begin{aligned} M_{u\text{temp}} &= 1.25 \cdot DC_{\text{temp}} + 1.5 \cdot LL_{\text{temp}} \\ &= 1.25 \cdot (4864) + 1.5 \cdot (618) = 7007 \text{ k-ft} \end{aligned}$$

For top flange:

$$f_{\text{buc}} = \frac{M_{u\text{temp}}}{S_t} = \frac{(7007) \cdot 12}{1981} = 42.4 \text{ ksi}$$

For bottom flange:

$$f_{\text{but}} = \frac{M_{u\text{temp}}}{S_b} = \frac{(7007) \cdot (12)}{2318} = 36.3 \text{ ksi}$$

The flange lateral bending stress,  $f_\ell$ , is assumed equal to 0.0 ksi.

The flow chart to evaluate the constructibility of the girder is shown in Appendix C6.4, Figure C6.4.1-1.

Begin by checking compression (top) flange. During construction, the top flange is discretely braced.

**[6.10.3.2.1]**

Check flange nominal yielding:

$$f_{buc} + f_{\ell} \leq \phi_f \cdot R_h \cdot F_{yc}$$

$$\phi_f = 1.0 \quad R_h = 1.0 \quad F_{yc} = 50.0 \text{ ksi}$$

$$\phi_f \cdot R_h \cdot F_{yc} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

$$f_{buc} + f_{\ell} = 42.4 + 0.0 = 42.4 \text{ ksi} < 50.0 \text{ ksi} \quad \underline{OK}$$

Check flexural resistance:

$$f_{buc} + \frac{1}{3} \cdot f_{\ell} \leq \phi_f \cdot F_{nc}$$

$$\phi_f = 1.0$$

$F_{nc}$  shall be taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance.

**[6.10.8.2.2]**

Local buckling resistance:

$$\lambda_f = \frac{b_{fc}}{2 \cdot t_{fc}} = \frac{20}{2 \cdot (1.0)} = 10.0$$

**[C6.10.8.2.2]**

$$\lambda_{pf} = 9.2, \text{ for } F_{yc} = 50.0 \text{ ksi}$$

Then,  $\lambda_f > \lambda_{pf}$  and compression flange is noncompact.

$$F_{nc(\text{FLB})} = \left[ 1 - \left( 1 - \frac{F_{yr}}{R_h \cdot F_{yc}} \right) \left( \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$$F_{yr} = \text{smaller of } \begin{cases} 0.7 \cdot F_{yc} = 0.7 \cdot (50.0) = 35.0 \text{ ksi} \\ F_{yw} = 50.0 \text{ ksi} \end{cases}$$

$$\text{But minimum } F_{yr} = 0.5 \cdot F_{yc} = 25.0 \text{ ksi}$$

Therefore,  $F_{yr} = 35.0 \text{ ksi}$

$$\lambda_{rf} = 0.56 \cdot \sqrt{\frac{E}{F_{yr}}} = 0.56 \cdot \sqrt{\frac{29,000}{35}} = 16.1$$

When checking constructability,  $R_b = 1.0$

The girder is homogeneous, so  $R_h = 1.0$

$$F_{nc(FLB)} = \left[ 1 - \left( 1 - \frac{35}{1.0 \cdot 50} \right) \left( \frac{10.0 - 9.2}{16.1 - 9.2} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0 = 48.3 \text{ ksi}$$

**[6.10.8.2.3]**

Lateral Torsional Buckling Resistance:

$$L_b = 26 \text{ ft} = 312 \text{ in}$$

$$L_p = 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}}$$

$$D_c = y_t - t_{fc} = 38.96 - 1.0 = 37.96 \text{ in}$$

$$r_t = \frac{b_{fc}}{\sqrt{12 \cdot \left( 1 + \frac{D_c t_w}{3 b_{fc} t_{fc}} \right)}} = \frac{20}{\sqrt{12 \cdot \left( 1 + \frac{37.96 \cdot 0.625}{3 \cdot 20 \cdot 1.0} \right)}} = 4.89 \text{ in}$$

$$L_p = 1.0 \cdot (4.89) \cdot \sqrt{\frac{29,000}{50.0}} = 117.8 \text{ in}$$

$$L_r = \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}} = \pi \cdot (4.89) \cdot \sqrt{\frac{29,000}{35.0}} = 442.2 \text{ in}$$

Therefore,  $L_p < L_b < L_r$

$$\text{Then } F_{nc(LTB)} = C_b \cdot \left[ 1 - \left( 1 - \frac{F_{yr}}{R_h \cdot F_{yc}} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_{yc}$$

First determine  $C_b$  :

$f_2$  is the compression stress at the third diaphragm away from the abutment:

$$f_2 = \frac{[(1.25)(4815) + 1.5(618)](12)}{1981} = 42.1 \text{ ksi}$$

$f_0$  is the compression stress at the second diaphragm away from the abutment:

$$f_0 = \frac{[(1.25)(4522) + (1.5)(566)] \cdot 12}{1981} = 39.4 \text{ ksi}$$

$f_{\text{mid}}$  is the compression stress at the midway point between the second and third diaphragm away from the abutment:

$$f_{\text{mid}} = \frac{[(1.25)(4832) + (1.5)(612)] \cdot 12}{1981} = 42.1 \text{ ksi}$$

$$f_{\text{mid}} = f_2$$

Therefore,  $C_b = 1.0$

$$F_{\text{nc(LTB)}} = 1.0 \cdot \left[ 1 - \left( 1 - \frac{35.0}{1.0 \cdot 50.0} \right) \left( \frac{312 - 117.8}{442.2 - 117.8} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0$$

$$= 41.0 \text{ ksi}$$

Then,  $F_{\text{nc}} = 41.0 \text{ ksi}$

$$\phi_f \cdot F_{\text{nc}} = 1 \cdot 41.0 = 41.0 \text{ ksi} < 42.4 \text{ ksi} \quad (3.4\% \text{ overstress})$$

The top flange compression stress is greater than 3% over the factored resistance. The 20 psf construction live load was placed over the entire first span for simplicity and to be conservative. Since this is an unlikely loading case and the overstress is only at 3.4%, by engineering judgement the flexural resistance is considered adequate.

Check web bend buckling. First, determine the nominal elastic web bend buckling resistance.

**[6.10.1.9.1]**

$$F_{\text{crw}} = \frac{0.9 \cdot E \cdot k}{\left( \frac{D}{t_w} \right)^2} \quad \text{not to exceed the smaller of } R_h F_{yc} \text{ and } F_{yw} / 0.7.$$

$$k = 9.0 \cdot \left( \frac{D}{D_c} \right)^2 = 9.0 \cdot \left( \frac{70}{37.96} \right)^2 = 30.6$$

Substituting values into Equation 6.10.1.9.1-1 results in

$$F_{\text{crw}} = \left[ \frac{0.9 \cdot 29,000 \cdot 30.6}{\left( \frac{70}{0.625} \right)^2} \right] = 63.7 \text{ ksi}$$

$$R_h F_{yc} = 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

$$\frac{F_{yw}}{0.7} = \frac{50.0}{0.7} = 71.4 \text{ ksi}$$

Set limit at  $F_{crw} = 50.0 \text{ ksi}$

**[6.10.3.2.1]**

$$\phi_f F_{crw} = 1.0 \cdot 50.0 = 50.0 \text{ ksi} > 42.4 \text{ ksi} \quad \underline{\text{OK}}$$

**[6.10.3.2.2]**

Now check tension (bottom) flange:

$$f_{bu} + f_\ell \leq \phi_f R_h F_{yt}$$

$$\phi_f = 1.0 \quad R_h = 1.0 \quad F_{yt} = 50.0 \text{ ksi}$$

$$\phi_f R_h F_{yt} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

Assuming  $f_\ell = 0.0 \text{ ksi}$  for constructibility check

$$f_{but} + f_\ell = 36.3 \text{ ksi} < 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

**H.2 Design  
Sections 2 & 3 -  
Negative Moment**

The sections for negative moment were checked following the same procedure and found to be adequate.

**I. Investigate the  
Service Limit State  
[6.10.4]**

Overload provisions control the amount of permanent deflection. Refer to the flow chart shown in LRFD Figure C6.4.2-1. The Service II load combination shall apply.

**Flange Stress Limitations**

Mn/DOT limits the maximum flange stress to  $F_y$  under the Strength Limit State. LRFD Article 6.10.4.2.2 limits the maximum flange stress to  $0.95 F_y$  under the Service II load combination:

$$1.0 f_{DC} + 1.3 f_{LL} \leq 0.95 F_y$$

Dividing through by 0.95,

$$1.05 f_{DC} + 1.37 f_{LL} \leq F_y$$

Compare the load factors above with those of the Strength Limit State. By inspection, you can see that the flanges will pass this check due to the smaller load factors associated with the Service II load combination.

### Bend Buckling Resistance

For the composite section in positive flexure,

[6.10.2.1.1]

$$\frac{D}{t_w} = \frac{70}{0.625} = 112 < 150 \quad \text{no checking of bend buckling required}$$

For the composite section in negative flexure, the compression flange stress due to the Service II Loads,  $f_c$ , shall satisfy LRFD Equation 6.10.4.2.2-4.

Compression stress of bottom flange at Section 3:

$$f_c = (1.0) \left( \frac{9231}{5437} + \frac{1660}{5631} \right) (12) + (1.3) \left( \frac{6905}{5631} \right) (12) (0.793) = 39.1 \text{ ksi}$$

Compression stress of bottom flange at Section 2:

$$f_c = (1.0) \left( \frac{4268}{3149} + \frac{749}{3349} \right) (12) + (1.3) \left( \frac{4041}{3349} \right) (12) (0.793) = 33.9 \text{ ksi}$$

[6.10.1.9.1]

$$F_{crw} = \frac{0.9 \cdot E \cdot k}{\left( \frac{D}{t_w} \right)^2}$$

For Section 3:

$$k = 9.0 \cdot \left( \frac{D}{D_c} \right)^2 = 9.0 \cdot \left( \frac{70}{36.47} \right)^2 = 33.2$$

Substituting values to arrive at the limiting stress results in

$$\left[ \frac{0.9 \cdot 29,000 \cdot 33.2}{\left[ \frac{70}{0.625} \right]^2} \right] = 69.1 \text{ ksi}$$

For Section 2:

$$k = 30.7 \text{ and } \frac{0.9 \cdot E \cdot k}{\left(\frac{D}{t_w}\right)^2} = 63.9 \text{ ksi}$$

The upper limit is capped by the smaller of

$$R_h F_{yc} = 50.0 \text{ and } \frac{F_{yw}}{0.7} = 71.4 \text{ ksi. So, } F_{crw} = 50.0 \text{ ksi.}$$

For Section 3:

$$f_c = 39.1 \text{ ksi} < F_{crw} = 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

For Section 2:

$$f_c = 33.9 \text{ ksi} < F_{crw} = 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

The web bend-buckling resistance is adequate.

***J. Investigate the Fatigue Limit State [6.10.5]***

Although LRFD 6.6.1.2.3 states that only details with fatigue resistance Category C or lower resistance need to be evaluated during design, details that are classified as Category B' and above should be checked.

***J.1 Fatigue Loading [3.6.1.4]***

The HL-93 truck is used to generate the fatigue loads that are used to evaluate different components of a design. For fatigue, the HL-93 truck has a fixed rear axle spacing of 30 feet. In addition, a load factor of 0.75 is applied to calibrate the stresses to those observed in field studies. The dynamic load allowance for fatigue loading is 15%. Distribution for fatigue is equal to the one design lane loaded distribution, with the multiple presence factor removed (if approximate equations are used for one lane loaded).

***[6.6.1.2.2]***

$$\gamma \cdot (\Delta f) \leq (\Delta F)_n$$

$$\gamma = \text{load factor for fatigue} = 0.75$$

$$(\Delta f) = \text{live load stress range}$$

$$(\Delta F)_n = \text{nominal fatigue resistance}$$

**J.2 Check Largest Stress Range Location**

The unfactored fatigue moments in Table 6.9.8 are multiplied by the fatigue load factor (0.75) and the appropriate distribution factor to arrive at the design moment ranges for fatigue. In Table 6.9.14 the stresses at the positive flexure section are computed by dividing the design moment range by the composite (n) section modulus assuming the deck is effective for both positive and negative moment.

Designers should note that the fatigue distribution factor for the exterior girder is significantly larger (0.676 versus 0.398/0.426) than that of the interior girders.

**Table 6.9.14  
Fatigue Range**

Girder Point	Fatigue			
	* Fatigue Moment Range Per Lane (kip-ft)	** Factored Design Moment Range (kip-ft)	Top Stress Range (kip)	Bottom Stress Range (kip)
0.0	0	0	0.00	0.00
0.1	1248	373	0.28	1.42
0.2	2136	638	0.48	2.44
0.3	2691	803	0.61	3.07
0.4	2978	889	0.67	3.40
0.5	3053	911	0.69	3.48
0.6	2959	883	0.67	3.38
0.691 <sup>(1)</sup>	2691	860	0.65	3.29
0.7	2659	850	0.61	2.57
0.8	2209	706	0.51	2.14
0.860 <sup>(2)</sup>	1908	610	0.44	1.85
0.9	1759	562	0.37	1.05
1.0	1567	501	0.33	0.94

\* Includes 15% Dynamic Load Allowance

\*\* Girder Point 0.1 – 0.691: (Fatigue Moment Range) × 0.75 × 0.398

Girder Point 0.691 – 1.0: (Fatigue Moment Range) × 0.75 × 0.426

<sup>(1)</sup> Field Splice

<sup>(2)</sup> Flange butt splice

**J.3 Check Fatigue Details**

**[6.6.1.2]**

**[Table 6.6.1.2.3-1]**

**[Table 6.6.1.2.5-3]**

For this example, the details that should be investigated for fatigue are: the welded flange butt splices, the web to flange welds, the toe of stiffener to web welds, the toe of stiffener to flange welds, and the shear stud to top flange welds. Fatigue at the bolted field splice should also be

investigated and will be considered later. Details subject to stress ranges less than  $1/2$  the infinite life fatigue threshold are assumed to have infinite life. The  $1/2$  factor accounts for the probability that some vehicles larger than the HL-93 fatigue truck will cross the bridge.

The worst case detail for fatigue is the shear stud to top flange weld which is a Category C detail. Category C details have a constant amplitude fatigue threshold  $(\Delta F)_{TH}$  of 10.0 ksi. From Table 6.9.14 the largest flange stress range is 3.48 ksi. This value is below  $1/2$  of the constant amplitude fatigue threshold (5.0 ksi). Therefore, all of the details have an infinite fatigue life.

#### ***J.4 Fatigue Requirements for Web***

##### ***[6.10.5.3]***

To control out-of-plane flexing of the web under repeated live loading the following constraints are placed on webs.

Interior panels of webs with transverse stiffeners, with or without longitudinal stiffeners, shall satisfy  $V_u \leq V_{cr}$ , where  $V_u$  is the maximum elastic shear stress in the web due to unfactored permanent load and factored fatigue load.

The live load used for this check is twice that presented in LRFD Table 3.4.1-1.

The computations for the shear buckling resistance of the web in shear is based on the following equation:

##### ***[6.10.9.3.3]***

$$V_{cr} = CV_p$$

$$V_p = 0.58 \cdot F_{yw} (D)t_w = 0.58 \cdot (50.0) \cdot (70)(0.625) = 1268.8 \text{ kip}$$

C is the ratio of shear buckling stress to shear yield strength.

Since the transverse stiffener spacing exceeds  $3D=17.5$  ft, the web is unstiffened.

$$\frac{D}{t_w} = \frac{70}{0.625} = 112$$

$$k = 5$$

##### ***[6.10.9.2]***

$$1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 1.40 \cdot \sqrt{\frac{29,000 \cdot 5.0}{50.0}} = 75.4 < \frac{D}{t_w}$$

Then,

[Eqn. 6.10.9.3.2-6]

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) = \frac{1.57}{(112)^2} \cdot \left(\frac{29,000 \cdot 5.0}{50.0}\right) = 0.363$$

$$V_{cr} = CV_p = 0.363 \cdot (1268.8) = 460.6 \text{ kip}$$

**Table 6.9.15**  
**Shear Fatigue**

Girder Point	DC1 Shear (kips)	DC2 Shear (kips)	Fatigue (per lane, no impact)		* V <sub>u</sub> (kips)
			Minimum Shear (ksi)	Maximum Shear (ksi)	
0.0	115	23		69	224
0.1	82	16		58	171
0.2	49	10		48	119
0.3	15	3		39	67
0.4	-18	-3	-30		-59
0.5	-51	-9	-38		-108
0.6	-85	-16	-47		-160
0.691 <sup>(1)</sup>	-116	-22	-54		-206
0.7	-119	-22	-56		-211
0.8	-154	-29	-64		-263
0.860 <sup>(2)</sup>	-176	-33	-68		-294
0.9	-191	-35	-70		-314
1.0	-230	-42	-76		-367

\* DC1 + DC2 + (Maximum Shear or Minimum Shear) × 0.725 × 0.75 × 2.0 × 1.15

<sup>(1)</sup> Field Splice

<sup>(2)</sup> Flange butt splice

The shear stresses at all girder points are well below the 460.6 kip permitted. The web satisfies the shear fatigue checks.

**K. Strength Limit State Shear Resistance [6.10.9]**

Within the commentary to LRFD Article 6.10.9.1 a flow chart identifies the steps for the shear design of I-sections. A copy of the flow chart is provided below in Figure 6.9.8.

[6.10.9.1]

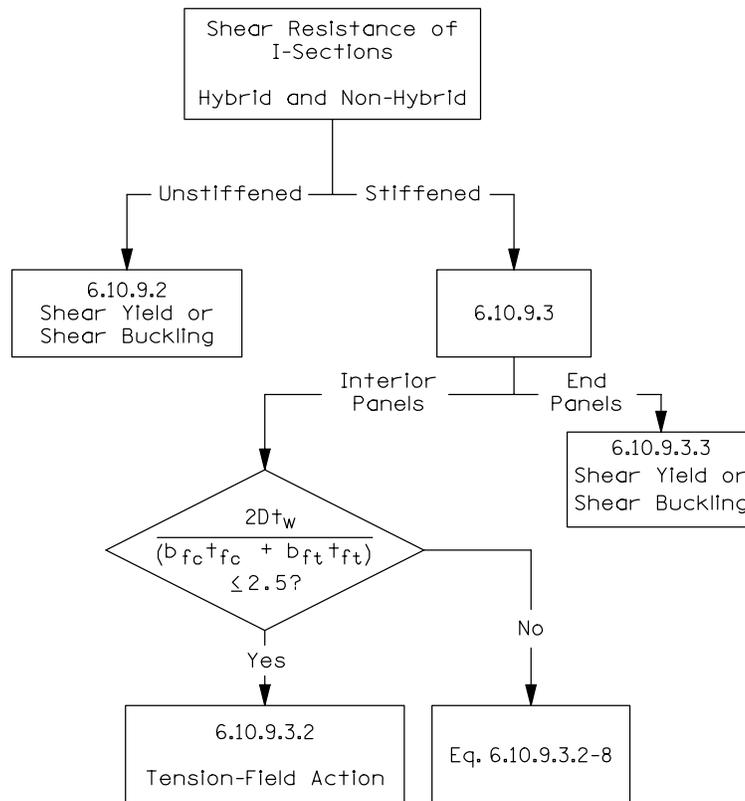


Figure 6.9.8

Determine the maximum shear capacity of the section with an unstiffened web and compare that to the required shear resistance.

[6.10.9.2]

$$V_n = C V_p$$

$$V_p = 0.58 \cdot F_{yw} \cdot (D)(t_w) = 0.58 \cdot (50.0) \cdot (70)(0.625) = 1268.8 \text{ kips}$$

C is dependent on the web depth to thickness ratio.

$$\frac{D}{t_w} = \frac{70}{0.625} = 112$$

[6.10.9.3.2]

The appropriate equation for C is selected based on how slender the web is:

for unstiffened web,  $k = 5.0$

$$1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 1.40 \cdot \sqrt{\frac{29,000 \cdot 5.0}{50.0}} = 75.4 < \frac{D}{t_w}$$

Then,

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) = \frac{1.57}{(112)^2} \cdot \left(\frac{29,000 \cdot 5.0}{50.0}\right) = 0.363$$

The capacity of the unstiffened web is:

$$V_n = 0.363 \cdot 1268.8 = 460.6 \text{ kips}$$

**[6.5.4.2]**

$$\phi_v = 1.0$$

$$V_r = \phi_v V_n = (1.0)(460.6) = 460.6 \text{ kips}$$

### ***K.1 Pier Region***

Assume the critical section for shear is at Girder Point 1.0. Based on Tables 6.9.3, 6.9.9, and 6.9.11, the factored shear force over the pier is:

$$V_{u(1.0)} = 1.25 \cdot (230 + 42) + 1.75 \cdot 166 \cdot 1.112 = 663 \text{ kips} > \phi V_n$$

The resistance of an unstiffened web is less than the demand of 663 kips; therefore, transverse stiffeners are required near the pier.

**[6.10.9.1]**

In order to qualify as a stiffened web, the maximum spacing for transverse stiffeners is three times the depth of the web:

$$3D = 3 \cdot (70) = 210 \text{ in} = 17.5 \text{ ft}$$

The diaphragms in the region of the pier are spaced at 22.5 feet. The diaphragm connection plates act as web stiffeners. Try adding a stiffener midway between the pier and the first diaphragm away from the pier.

Then  $d_o = 11.25 \text{ ft} = 135 \text{ in}$

**[6.10.9.3.2]**

Compute  $k$ :

$$k = 5 + \frac{5}{\left[\frac{d_o}{D}\right]^2} = 5 + \frac{5}{\left[\frac{135}{70}\right]^2} = 6.3$$

$$1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 1.40 \cdot \sqrt{\frac{29,000 \cdot 6.3}{50.0}} = 84.6 < 112$$

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) = \frac{1.57}{\left(\frac{70}{0.625}\right)^2} \cdot \left(\frac{29,000 \cdot 6.3}{50.0}\right) = 0.457$$

For an interior panel at Girder Point 1.0,

$$\frac{2D \cdot t_w}{(b_{fc} \cdot t_{fc}) + (b_{ft} \cdot t_{ft})} = \frac{2(70)(0.625)}{(22)(3.25) + (20)(3.25)} = 0.64 < 2.5$$

Then,

$$V_n = V_p \cdot \left[ C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \right]$$

$$= 1268.8 \cdot \left[ 0.457 + \frac{0.87 \cdot (1 - 0.457)}{\sqrt{1 + \left(\frac{135}{70}\right)^2}} \right] = 855.8 \text{ kips}$$

$$V_r = \phi_v V_n = (1.0) \cdot (855.8) = 855.8 \text{ kips} > 663 \text{ kips} \quad \underline{\text{OK}}$$

Check if an additional stiffener is required midway between the first and second diaphragm away from the pier by checking the shear at the first diaphragm (0.871 girder point).

$$V_{u(0.871)} = 1.25 \cdot (180 + 33) + 1.75 \cdot 144 \cdot 1.112 = 546.5 \text{ kips} > 460.6 \text{ kips}$$

Therefore an additional stiffener is needed.

Check if another stiffener is needed between the second and third diaphragm by checking the shear at the second diaphragm (0.742 girder point).

$$V_{u(0.742)} = 1.25 \cdot (134 + 25) + 1.75 \cdot (122) \cdot 1.112$$

$$= 436.2 \text{ kips} < 460.6 \text{ kips} \quad \underline{\text{OK}}$$

Therefore, stiffeners are not required between the second and third diaphragms away from the pier.

### ***K.2 Abutment Region***

From previous calculations, the capacity of the unstiffened web is  
 $V_r = 460.6 \text{ kips}$

#### ***[6.10.9.2-1]***

At Girder Point 0.0 the shear demand is:

$$V_{u(0.0)} = 1.25 \cdot (115 + 23) + 1.75 \cdot (137) \cdot 1.112 = 439.1 < 460.6 \text{ kips}$$

The web has adequate capacity at the abutment without stiffeners.

### ***K.3 Transverse Stiffener Design*** ***[6.10.11.1]***

Ideally the size of the stiffener should be coordinated with the cross frame connection plates. Fabrication of the girder will be simplified if only one plate size and thickness is welded to the web at non-bearing locations. In addition, transverse stiffeners and diaphragm connection plates should be detailed with widths that are in  $\frac{1}{4}$  inch increments. This provides the fabricator additional flexibility. They can either cut the stiffeners and connection plates out of large mill plate or utilize standard flat bar stock.

Transverse stiffeners are required near the pier. Mn/DOT Detail B411 (Stiffener Details) addresses the constraints placed on stiffeners in LRFD Article 6.10.11.1.1.

The dimensions of transverse stiffeners are required to fall within geometric constraints based on section depth, flange width, and projecting element thickness.

#### ***[6.10.11.1.2]***

Begin with the projecting width constraint:

$$b_t \geq 2.0 + \frac{D}{30} = 2.0 + \frac{70}{30} = 4.33 \text{ in}$$

Try a single 8" x  $\frac{1}{2}$ " stiffener.

$$b_t = 8 \text{ in} \qquad t_p = 0.50 \text{ in}$$

Check flange width constraint:

$$16.0 \cdot t_p = 16.0 \cdot 0.50 = 8.0 \text{ in} \quad \underline{\text{OK}}$$

$$0.25 \cdot b_f = 0.25 \cdot (22) = 5.5 \text{ in} < 8.0 \text{ in} \quad \underline{\text{OK}}$$

**[6.10.11.1.3]**

In addition to good aspect ratios, stiffeners must also have an adequate moment of inertia.

$$J = 2.5 \cdot \left( \frac{D}{d_o} \right)^2 - 2.0 = 2.5 \cdot \left( \frac{70}{135} \right)^2 - 2.0 = -1.33 < 0.5$$

Therefore,  $J = 0.5$ .

For a transverse stiffener spacing of 135 inches, the shear buckling resistance of the web  $V_{cr}$  is:

$$V_{cr} = CV_p = 0.457 \cdot 1268.8 = 579.8 \text{ kips} < 663 \text{ kips}$$

Because the factored shear force  $V_u$  (1.0) is greater than  $V_{cr}$ , the required stiffness  $I_t$  of the stiffeners shall satisfy:

$$\text{Min. } I_t \geq \frac{D^4 \rho_t^{1.3}}{40} \cdot \left( \frac{F_{yw}}{E} \right)^{1.5}$$

$$\rho_t = \text{larger of } \begin{cases} F_{yw}/F_{crs} \\ 1.0 \end{cases}$$

$$F_{crs} = \frac{0.31 \cdot E}{\left( \frac{b_t}{t_p} \right)^2} \leq F_{ys} = \frac{0.31 \cdot 29,000}{\left( \frac{8}{0.5} \right)^2} = 35.1 \text{ ksi} < F_{ys} = 50 \text{ ksi}$$

$$\frac{F_{yw}}{F_{crs}} = \frac{50}{35.1} = 1.42$$

$$\rho_t = 1.42$$

$$\text{Min. } I_t \geq \frac{(70.0)^4 \cdot (1.42)^{1.3}}{40.0} \cdot \left( \frac{50.0}{29,000} \right)^{1.5} = 67.8 \text{ in}^4$$

The stiffener moment of inertia taken about the edge in contact with the web is:

$$\text{Actual } I_t = \frac{1}{3} \cdot 0.5 \cdot 8^3 = 85.3 > 67.8 \text{ in}^4 \quad \underline{\text{OK}}$$

**K.4 Bearing  
Stiffener Design  
[6.10.11.2.1]**

For welded plate girders, bearing stiffeners are needed at both the abutments and piers.

Abutment Bearing

The reaction to be carried by the bearing stiffeners is:

$$R_u = 1.25 \cdot (115 + 23) + 1.75 \cdot (137) \cdot 1.112 = 439 \text{ kips}$$

Similar to transverse stiffeners, there are constraints on the geometry of bearing stiffeners.

The bearing stiffeners should extend close to the outside edges of the narrower flange, which is 20 inches in width. Try a 1" x 9" wide bearing stiffener on each side of the web.

Begin by checking the projecting width.

**[6.10.11.2.2]**

$$0.48 \cdot t_p \cdot \sqrt{\frac{E}{F_{ys}}} = 0.48 \cdot (1.0) \cdot \sqrt{\frac{29,000}{50}} = 11.56 \text{ in} > 9.00 \text{ in} \quad \underline{\text{OK}}$$

The bearing resistance check is based on the net area of steel in contact with the flange. Assume a 1<sup>1</sup>/<sub>2</sub> inch cope at the bottom of the stiffener in accordance with Mn/DOT Detail B411.

**[6.10.11.2.3]**

$$(R_{sb})_n = 1.4 \cdot A_{pn} \cdot F_{ys} = 1.4 \cdot [1.00 \cdot (9.0 - 1.5) \cdot 2] \cdot (50.0) = 1050 \text{ kips}$$

$$\phi_b \cdot (R_{sb}) = 1.0 \cdot (1050) = 1050 \text{ kips} > 439 \text{ kips} \quad \underline{\text{OK}}$$

**[6.10.11.2.4]**

Now check the axial resistance of the bearing stiffeners.

**[6.10.11.2.4b]**

The stiffeners will act like a column while supporting the bearing reaction. The effective section consists of the stiffeners, plus  $9t_w$  (thickness of the girder web) on each side of the stiffeners (see Figure 6.9.9).

The area for this column is:

$$A = 1.0 \cdot 9.0 \cdot 2 + 11.25 \cdot 0.625 = 25.03 \text{ in}^2$$

The moment of inertia about the girder web is:

$$I = \frac{1}{12} \cdot (11.25 - 1.0) \cdot 0.625^3 + \frac{1}{12} \cdot 1.0 \cdot 18.625^3 = 538.6 \text{ in}^4$$

The radius of gyration is:

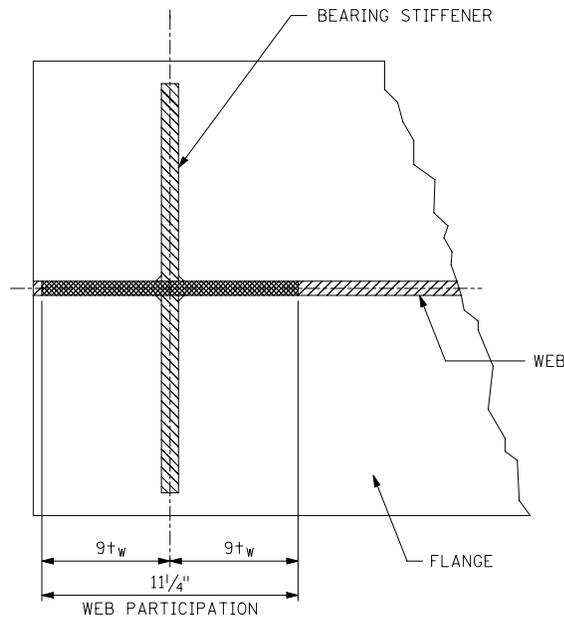
$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{538.6}{25.03}} = 4.64 \text{ in}$$

Check the width/thickness limits of Article 6.9.4.2

**[6.9.4.2]**

$$k \cdot \sqrt{\frac{E}{F_y}} = 0.45 \cdot \sqrt{\frac{29,000}{50.0}} = 10.84$$

$$\frac{b}{t} = \frac{9.0}{1.0} = 9.0 < 10.84 \quad \text{OK}$$



**Figure 6.9.9**  
**"Column" for Bearing Stiffener at Abutment**

[6.9.3]

Check slenderness ratio:

[6.10.11.2.4a]

The effective length ( $K\ell$ ) of the column is  $0.75D = 0.75 \cdot (70) = 52.5$  in

$$\frac{K\ell}{r} = \frac{52.5}{4.64} = 11.3 < 120 \quad \underline{\text{OK}}$$

[6.9.4.1]

Determine factored axial resistance:

$$\lambda = \left( \frac{K\ell}{r \cdot \pi} \right)^2 \cdot \frac{F_{ys}}{E} = \left( \frac{52.5}{4.64 \cdot \pi} \right)^2 \cdot \frac{50.0}{29,000} = 0.02$$

Since  $\lambda < 2.25$ ,

$$P_n = 0.66^\lambda \cdot F_y \cdot A_s = 0.66^{0.02} \cdot (50.0) \cdot (25.03) = 1241 \text{ kips}$$

[6.9.2.1]

$$P_r = \phi_c P_n = (0.90) \cdot (1241) = 1117 \text{ kips} > 439 \text{ kips} \quad \underline{\text{OK}}$$

Therefore, use a pair of 1" x 9" bearing stiffeners at the abutments. Using the same design procedure, a pair of 1<sup>1</sup>/<sub>2</sub>" x 9" bearing stiffeners were found adequate to carry the factored pier reaction of 1341 kips.

**K.5 Shear  
Resistance During  
Construction**  
[6.10.3.3]

The web is to be investigated for the sum of factored permanent loads and factored construction loads applied to the non-composite section during construction. The web shall satisfy  $V_u < \phi_v V_{cr}$ . The normal shear resistance for this check is limited to the shear yielding or shear buckling resistance per Article 6.10.9.3.3. Using the same procedure used above, calculations show that the web has adequate capacity during construction.

**L. Design Shear  
Connectors**  
[6.10.10]

Shear connectors are to be placed along the full length of the girder, including negative moment regions, because the girder is designed as composite for negative moment.

Shear connectors are designed to satisfy fatigue constraints after which a strength check is performed. Assume that <sup>7</sup>/<sub>8</sub> inch diameter shear connectors will be used.

The minimum transverse spacing for connectors is 4.0 stud diameters. For <sup>7</sup>/<sub>8</sub> inch diameter studs, this translates into a minimum spacing of

3<sup>1</sup>/<sub>2</sub> inches. The minimum clear distance from a stud to the edge of a flange is 1.0 inch. With a 20 inch top flange width, the maximum number of stud spaces placed in a line across the flange is:

$$\frac{20 - 2 \cdot (1) - 0.875}{3.5} = 4.9 \text{ spaces}$$

Five studs across the flange is permissible, but use four shear studs at each location.

The studs must extend a minimum of 2 inches into the deck and have a minimum of 3 inches of cover. At midspan, the amount of concrete stool is 1.75 inches. At the pier, the amount of concrete stool is 2 inches. Choose a stud height of 5 inches.

**L.1 Fatigue Limit  
State**

**[6.10.10.1.2]**

**[6.10.10.2]**

The pitch P (longitudinal spacing) of each set of studs shall satisfy:

$$\text{Max } p \leq \frac{n \cdot Z_r}{V_{sr}}$$

The shear fatigue resistance of an individual connector is based on the number of fatigue cycles anticipated:  $Z_r = \alpha d^2 \geq \frac{5.5}{2} d^2$ .

where  $\alpha = 34.5 - 4.28 \cdot \log N$

The lower bound corresponds to the resistance for a stud subjected to approximately 26,200,000 cycles.

For sections near pier:

$$\text{ADTT} = 2000 \quad \text{Design Life} = 75 \text{ years}$$

From LRFD Table 6.6.12.5-2, use 1.5 cycles per truck passage.

Then

$$N = 1.5 \cdot 2000 \cdot 365 \cdot 75 = 82,125,000 \text{ cycles} > 26,200,000 \text{ cycles}$$

The lower bound governs:

$$Z_r = \frac{5.5}{2} \cdot d^2 = \frac{5.5}{2} \cdot 0.875^2 = 2.11 \text{ kips}$$

For sections away from pier:

From Table 6.6.12.5-2, 1.0 cycles per truck passage shall be used.

$$N = 1.0 \cdot 2000 \cdot 365 \cdot 75 = 54,750,000 \text{ cycles} > 26,200,000 \text{ cycles}$$

The lower bound governs:

$$Z_r = 2.11 \text{ kips}$$

$V_{sr}$  is to be computed as follows:

$$V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2}$$

For a straight span,  $F_{fat}$  may be taken as 0.

$$\text{Then } V_{sr} = V_{fat} = \frac{VQ}{I}$$

The inertia values are taken from Table 6.9.2:

For the positive moment region,  $I = 189,316 \text{ in}^4$ .

For the negative moment region,  $I = 132,855 \text{ in}^4$  (value for the smaller negative moment section).

Now compute the "Q" values. For the positive moment region:

$$Q = \frac{1}{n} \cdot b_{eff} \cdot t_s \cdot \left( y_{tc} + t_{cstool} + \frac{t_s}{2} \right) = \frac{1}{8} \cdot 118 \cdot 9 \cdot \left( 11.92 + 1.75 + \frac{9}{2} \right) = 2412 \text{ in}^3$$

For the negative moment region, only the area of steel in the concrete deck is considered.

$$d_{r_{avg}} = \frac{(7.80)(5.5) + (3.93)(1.88)}{(7.80 + 3.93)} = 4.29 \text{ inches from bottom of deck}$$

$$Q = (A_{rt} + A_{rb}) \left( y_{tc} + t_{cstool} + d_{r_{avg}} \right) = (7.80 + 3.93) (33.83 + 1.75 + 4.29) \\ = 468 \text{ in}^3$$

Knowing  $n$ ,  $Z_r$ ,  $I$ , and  $Q$  leaves the pitch to be a function of the fatigue shear force range  $V_f$ . For the positive moment region

$$\text{Max } p \leq \frac{n \cdot Z_r \cdot I}{V_f \cdot Q} = \frac{4 \cdot 2.11 \cdot 189,316}{V_f \cdot 2412} = \frac{662}{V_f}$$

For the negative moment region the required pitch is

$$\text{Max } p \leq \frac{n \cdot Z_r \cdot I}{V_f \cdot Q} = \frac{4 \cdot 2.11 \cdot 132,855}{V_f \cdot 468} = \frac{2396}{V_f}$$

**Table 6.9.16**  
**Shear Connector Spacing For Fatigue**

Girder Point	* Fatigue Shear LLDF	** Factored Fatigue Shear Range $V_f$ (kips)	Max p (Positive) (in)	Max p (Negative) (in)	*** Max p Limit (in)
0.0	0.725	42	16		24
0.1	0.725	36	18		24
0.2	0.725	31	21		24
0.3	0.725	31	22		24
0.4	0.725	31	21		24
0.5	0.725	33	20		24
0.6	0.725	34	20		24
0.7	0.725	35	19		24
0.8	0.725	37		64	24
0.9	0.725	39		61	24
1.0	0.725	41		58	24

\* See Table 6.9.3

\*\*  $0.75 \cdot \text{LLDF} \cdot \text{Fatigue Truck Shear Range}$  from Table 6.9.11

\*\*\* Per LRFD 6.10.10.1.2, the maximum limit for spacing of shear connectors is 24 inches and minimum limit is  $6_d = 6 \cdot 0.875 = 5.25$  in

By inspection, the negative moment fatigue requirements are satisfied if 2 studs are placed on a 20 inch spacing.

**L.2 Strength Limit State**  
**[6.10.10.4]**

In addition to fatigue, adequate studs are needed to ensure that the cross sections can generate the flexural resistance computed earlier.

The factored shear resistance of a single shear connector  $Q_r$ , shall be taken as:

$$Q_r = \phi_{sc} Q_n$$

$$\phi_{sc} = \text{resistance factor} = 0.85$$

**[6.5.4.2]**

**[6.10.10.4.3]**

The nominal resistance of a shear connector  $Q_n$  is:

$$Q_n = 0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c} = 0.5 \cdot 0.60 \cdot \sqrt{4 \cdot 3644} = 36.2 \text{ kips}$$

$$\text{But, } Q_n \leq A_{sc} \cdot F_u = 0.60 \cdot 60 = 36 \text{ kips}$$

$$Q_r = \phi Q_n = 0.85 \cdot 36 = 30.6 \text{ kips}$$

The minimum number of shear connectors,  $n$ , shall be taken as:

**[6.10.10.4.1]** 
$$n = \frac{P}{Q_r}$$

where,  $P$  = total nominal shear force (Article 6.10.10.4.2).

**[6.10.10.4.2]** For the region between the point of maximum positive LL + I moment and the abutment:

$$P = \sqrt{P_p^2 + F_p^2}$$

$F_p$  may taken as 0.0 for straight spans.

$P_p$  is taken as the lesser of the capacity of the deck or the capacity of the steel section.

The capacity of the deck is:

$$P_{1P} = 0.85 \cdot f'_c \cdot b_s \cdot t_s = 0.85 \cdot 4 \cdot 118 \cdot 9 = 3611 \text{ kips}$$

The capacity of the steel section is:

$$\begin{aligned} P_{2P} &= F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc} \\ &= 50 \cdot (70 \cdot 0.625 + 22 \cdot 1.25 + 20 \cdot 1.0) = 4563 \text{ kips} \end{aligned}$$

So,  $P_p = P_{1P} = 3611 \text{ kips}$

$$P = P_p = 3611 \text{ kip}$$

$$n = \frac{P}{Q_r} = \frac{3611}{30.6} = 118 \text{ studs}$$

For the region between the point of maximum positive LL + I moment and the centerline of an adjacent interior support:

$$P = \sqrt{P_T^2 + F_T^2}$$

$F_T$  may be taken as 0.0 for straight spans.

$$P_T = P_p + P_n$$

$P_n$  is total longitudinal shear force in the concrete deck over an interior support taken as the lesser of either:

$$\begin{aligned} P_{1n} &= F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc} \\ &= 50 \cdot (70 \cdot 0.625 + 20 \cdot 3.25 + 22 \cdot 3.25) = 9013 \text{ kips} \end{aligned}$$

or

$$P_{2n} = 0.45 \cdot f'_c \cdot b_s \cdot t_s = 0.45 \cdot 4.0 \cdot 118 \cdot 9.0 = 1912 \text{ kips}$$

So,  $P_n = P_{2n} = 1912 \text{ kips}$

$$P_T = P_n + P_p = 1912 + 3611 = 5523 \text{ kips}$$

$$P = P_T = 5523 \text{ kips}$$

$$n = \frac{P}{Q_r} = \frac{5523}{30.6} = 180 \text{ studs}$$

The final details for the shear studs need to satisfy the constraints of both the fatigue design and the strength design. After reviewing the constraints, the layout provided in Figure 6.9.16 was chosen.

***M. Investigate the Field Splice Design [6.13]***

Several items need to be considered when locating and designing field splices for steel girders. Typically, splices are located near inflection points to minimize the flexural resistance required of the connection. In addition, designers need to ensure that adequate clearance is provided to transverse stiffeners, cross frame connection plates, etc.

As a general rule, designers should limit the number of plate thicknesses used in a splice. The splice used for this example has four plate thicknesses ( $\frac{3}{8}$ ",  $\frac{1}{2}$ ",  $\frac{5}{8}$ ", and  $\frac{3}{4}$ ") used for the splice and fill plates.

The number of limit states and loading conditions to consider in the design of a splice is significant. Construction, Service II (permanent deflection), Fatigue, and Strength limit states should all be evaluated. In most cases, the Strength limit state will dictate the plate sizes and the

number of bolts. The bolted connections used in the splice are Category B details.

Typically, three splice plates are used for each flange and two splice plates are used for the web. This permits all of the bolts to function in double shear and minimizes the number of bolts required.

As the size of splice plates are considered, it is prudent to look at the change in plate sizes on both sides of the splice. The thickness of fill plates can be determined prior to any design of the connection. For this example, the top flange on the left is 1" x 20" and on the right is 1<sup>3</sup>/<sub>4</sub> x 20". The fill plate for the top flange splice will have a thickness of 3/4". The web on both sides of the splice is 5/8". No fill plate will be necessary for the web. The bottom flange on the left is 1<sup>1</sup>/<sub>4</sub>" x 22". The bottom flange on the right is 1<sup>3</sup>/<sub>4</sub>" x 22". A 1/2" fill plate will be required for the bottom flange splice. Using splice plates with a 3/4" or 1/2" thickness will minimize the number of plate thicknesses required for the splice.

The splice will be designed as a slip-critical connection. The bolted connections will be proportioned:

- 1) to provide shear and bearing resistance under the governing strength limit state
- 2) to prevent slip at the Service II limit state
- 3) to have adequate fatigue resistance

The resistance of the bolts will be designed based on threads excluded from the shear plane for plates which are 3/8 inch thick or greater.

The loads at the location of the splice are shown in Table 6.9.17.

**Table 6.9.17**  
**Loads at Girder Point 0.69 (Unfactored)**

Component	Moment (k-ft)	Shear (k)
DC1	18	-116
DC2	54	-22
Pos. M LL + I	2351	22
Neg. M LL + I	-2469	-126
Pos. M DC <sub>CONST</sub>	2416	-98
Neg. M DC <sub>CONST</sub>	-2369	49
Pos. M LL <sub>CONST</sub>	369	-11
Neg. M LL <sub>CONST</sub>	-371	3
Fatigue LL + I Range	2691	64

The loads are applied to the non-composite, short-term composite ( $n$ ), and long-term composite ( $3n$ ) cross sections. Table 6.9.18 shows the section properties used for the splice design.

To arrive at design stresses for the splice plates, the loads are applied to the appropriate section. The stresses from the load components are then factored to arrive at design stresses. Table 6.9.19 lists the unfactored component stresses and the factored design stresses for the flanges and the web. Flange splices are based on mid-flange stresses. Web splices can conservatively be based on mid-flange stresses or can use the stresses at the top and bottom of the web. The strength of the splice is based on the capacity of the smaller girder framing into the connection. For this example, the positive moment section is the smaller capacity member.

**Table 6.9.18**  
**Section Properties for Splice Design**

Parameter	Design Section 1			
	Non-composite	Positive Moment		Neg. Moment
		Long-Term Composite ( $3 \cdot n$ )	Short-Term Composite ( $n$ )	
Moment of Inertia ( $\text{in}^4$ )	77,179	139,281	189,316	98,220
$y_t$	38.96	23.93	11.92	33.83
$y_b$	33.29	48.32	60.33	38.42
Top Flange Thickness (in)	1	1	1	1
Bottom Flange Thickness (in)	1.25	1.25	1.25	1.25
$S_{\text{mid}}$ top flange	2007	5945	16,578	2947
$S_{\text{top}}$ web	2033	6074	17,337	2992
$S_{\text{bottom}}$ web	2409	2959	3204	2642
$S_{\text{mid}}$ bottom flange	2363	2920	3171	2599

**Table 6.9.19**  
**Flexural Stress Components at Splice**

Loading	Stress at Mid-depth of <sup>(1)</sup> Top Flange (ksi)	Stress at Mid-depth of <sup>(1)</sup> Bottom Flange (ksi)
DC1	-0.11	0.09
DC2	-0.11	0.22
Pos. M (LL + I)	-1.70	8.90
Neg. M (LL + I)	10.05	-11.40
Pos. M DC <sub>const</sub>	-14.45	12.27
Neg. M DC <sub>const</sub>	14.16	-12.03
Pos. M LL <sub>const</sub>	-2.21	1.87
Neg. M LL <sub>const</sub>	2.22	-1.88
Pos. M Strength I	-3.25	15.96
Neg. M Strength I	17.39	-19.67
Pos. M Service II	-2.43	11.88
Neg. M Service II	12.85	-14.51
Pos. M Service II Const.	-17.32	14.70
Neg. M Service II Const.	17.05	-14.47
Fatigue LL + I Range <sup>(2)</sup>	0.65	3.29

<sup>(1)</sup> Positive number denotes tension stress.

<sup>(2)</sup> Factored stress range.

### ***M.1 Controlling Flange*** ***[C6.13.6.1.4c]***

At the strength limit state, the controlling flange is defined as the flange with the maximum ratio of factored flexure stress to factored resistance. Table 6.9.19 indicates that the bottom flange is the controlling flange for both positive and negative live load of the Strength I Limit State. The splice must be capable of resisting both positive and negative live load moment conditions.

### **Bottom Flange Splice Plates**

When the combined area of the inner splice plates is within 10 percent of the area of the outer splice plate, both the inner and outer splice plate may be designed for one-half the flange design force. In addition, the area of the outer plate and the sum area of the inner plates each need to have a cross sectional area which is approximately half that of the flange. Try an outer splice plate that is  $\frac{5}{8}$ " x 22". Try inner splice plates that are  $\frac{3}{4}$ " x 10".

Gross area of inner splice plates:

$$A_{g\_in} = 2 \cdot 0.75 \cdot 10 = 15.00 \text{ in}^2$$

Gross area of outer splice plate:

$$A_{g\_out} = 0.625 \cdot 22 = 13.75 \text{ in}^2$$

The difference in area is 8%. Therefore equal distribution of the flange force to the splice plates can be assumed. Note that if the areas of the inner and outer splice plates had differed by more than 10%, the splice plate design force would be calculated by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the splice plates.

**[6.13.6.1.4c]**

Load Case I - Positive Live Load for Strength I Limit State

Bottom Flange is in tension.

The flange splice shall be designed to provide a minimum design resistance equal to the greater of:

$$F_{cf} = \frac{\left[ \frac{f_{cf}}{R_h} + \alpha \cdot \phi_f \cdot F_{yf} \right]}{2}$$

or

$$F_{cf} = 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf}$$

From Table 6.9.19

$$f_{cf} = 15.96 \text{ ksi}$$

$$F_{cf} = \frac{\left[ \frac{15.96}{1} + 1 \cdot 1 \cdot 50 \right]}{2} = 33.0 \text{ ksi}$$

or

$$F_{cf} = 0.75 \cdot 1 \cdot 1 \cdot 50 = 37.50 \text{ ksi} \quad \underline{\text{GOVERNS}}$$

Design force,  $T_{tcfdes} = F_{cf} \cdot A_e$

Bottom flange is a tension flange under positive live load moment.

$$A_e = \left( \frac{\phi_u F_u}{\phi_y F_{yt}} \right) A_n$$

where,

$$\phi_u = 0.8$$

$$F_u = 70.0 \text{ ksi}$$

$$\phi_y = 0.95$$

$$F_y = 50 \text{ ksi}$$

7/8" diameter bolts will be used for the splice design. For  $A_n$  calculation, assume 6 bolts per row with a 1" diameter.

$$A_n = (1.25)(22 - 6 \cdot 1) = 20.00 \text{ in}^2$$

$$A_e = \frac{(0.8)(70)}{(0.95)(50)} (20.00) = 23.58 \text{ in}^2 < A_g = 27.5 \text{ in}^2$$

$$T_{\text{tcfdes}} = (37.50) (23.58) = 884.3 \text{ kips}$$

Maximum Tension Design force for inner plates and outer plate:

$$T_{\text{in\_tcfdes}} = T_{\text{out\_tcfdes}} = \frac{884.3}{2} = 442.2 \text{ kips}$$

**[6.8.2.2]**

The factored tensile resistance,  $P_r$ , shall be taken as the lesser of the following:

$$\text{Yield on the gross section: } P_{r1} = \phi_y F_y A_g$$

**[6.13.5.2]**

$$\text{Fracture on the net section: } P_{r2} = \phi_u F_u A_n U \text{ where } A_n < 0.85 A_g \text{ and } U = 1$$

For Outer Splice Plate:

$$A_g = (22) (0.625) = 13.75 \text{ in}^2$$

Assuming 6 holes per row with a 1" diameter:

$$A_n = [22 - (6) (1)] (0.625) = 10.00 \text{ in}^2 < 0.85 \cdot A_g = 11.69 \text{ in}^2$$

$$P_{r1} = \phi_y \cdot F_y \cdot A_g = (0.95) (50) (13.75) = 653.1 \text{ kips}$$

$$P_{r2} = \phi_u \cdot F_u \cdot A_n \cdot U = (0.8) (70) (10.00) (1) = 560.0 \text{ kips} \quad \underline{\text{GOVERNS}}$$

$$P_r = 560.0 \text{ kips} > 442.2 \text{ kips} \quad \underline{\text{OK}}$$

For Inner Splice Plates:

$$A_g = (2) (10) (0.75) = 15.00 \text{ in}^2$$

$$A_n = (2) [10 - (3) (1)] (0.75) = 10.50 \text{ in}^2 < 0.85 \cdot A_g = 12.75 \text{ in}^2$$

$$P_{r1} = \phi_y \cdot F_y \cdot A_g = (0.95) (50) (15) = 712.5 \text{ kips}$$

$$P_{r2} = \phi_u \cdot F_u \cdot A_n \cdot U = (0.8) (70) (10.50) (1) = 588.0 \text{ kips} \quad \underline{\text{GOVERNS}}$$

$$P_r = 588.0 \text{ kips} > 442.2 \text{ kips} \quad \underline{\text{OK}}$$

**[6.13.6.1.4c]**

Load Case 2 - Negative Live Load at Strength I Limit State

Bottom Flange is in compression.

The flange splice shall be designed to provide a minimum design resistance equal to the greater of:

$$F_{cf} = \frac{\left[ \frac{|f_{cf}|}{R_n} + \alpha \phi_f F_{yf} \right]}{2}$$

or

$$F_{cf} = 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf}$$

From Table 6.9.19,  $f_{cf} = -19.67 \text{ ksi}$

$$F_{cf} = \frac{\left[ \frac{-19.67}{1.0} + (1.0) (1.0) (50.0) \right]}{2} = 34.8 \text{ ksi}$$

or

$$F_{cf} = (0.75) (1.0) (1.0) (50.0) = 37.5 \text{ ksi} \quad \underline{\text{GOVERNS}}$$

Design force:  $T_{cfdes} = F_{cf} \cdot A_e$

$$A_e = A_g \text{ for compression flange}$$

$$T_{cfdes} = (37.5) (22) (1.25) = 1031.3 \text{ kips}$$

Since the combined area of the inner splice plates is within 10% of the area of the outer splice plate, both the inner and outer splice plate can be designed for one-half the flange design force.

Maximum Compression design force for inner plate and outer plate is taken as:

$$T_{in\_ccfdes} = T_{out\_ccfdes} = \frac{1031.3}{2} = 515.7 \text{ kips}$$

The factored resistance of splice plates subjected to compression,  $R_r$ , shall be taken as:

$$R_r = \phi_c F_y A_s$$

Where:

$A_s$  = gross area of the splice plate

$$\phi_c = 0.9$$

For Outer Splice Plate:

$$R_r = (0.9) (50) (22) (0.625) = 618.8 \text{ kips} > 515.7 \text{ kips} \quad \underline{OK}$$

For Inner Splice Plate:

$$R_r = (0.9) (50) (10.0) (0.75) (2) = 675.0 \text{ kips} > 515.7 \text{ kips} \quad \underline{OK}$$

Load Case 3 - Fatigue

The fatigue detail category for a bolted connection is Category B. The splice is to be designed for infinite fatigue life and the Category B constant amplitude fatigue threshold  $(\Delta F)_{TH}$  is 16 ksi.

$$\text{Then } \Delta F_n = 0.5 \cdot (\Delta F)_{TH} = 0.5 \cdot 16 = 8.0 \text{ ksi}$$

$$\text{Actual range } \gamma \cdot (\Delta f) = 3.29 \text{ ksi} < 8.0 \text{ ksi} \quad \underline{OK}$$

### Bolt Shear Resistance

Now compute the resistance of a  $7/8$ " diameter A325 bolt and determine the number of bolts for the bottom flange splice plate. The filler plate for the bottom flange is  $1/2$  inch. Consequently, the fillers need to be extended or the capacity of the bolts reduced. For this example, the capacity of the bolts will be reduced (using LRFD Equation 6.13.6.1.5-1). Use a filler plate that is as wide as the flange.  $A_p$  is the smaller of:

$$A_p = (1.25) (22) = 27.5 \text{ in}^2 \quad \underline{GOVERNS}$$

[6.13.6.1.5]

or

$$A_p = (2)(10)(0.75) + (22)(0.625) = 28.75 \text{ in}^2$$

Fill plate reduction factor:

$$\gamma = \frac{A_r}{A_p} = \frac{(22)(0.5)}{28.75} = 0.40$$

$$R = \left[ \frac{(1 + \gamma)}{(1 + 2 \cdot \gamma)} \right] = \left[ \frac{(1 + 0.40)}{(1 + 2 \cdot 0.40)} \right] = 0.78$$

**[6.13.2.7]**

The shear resistance of a  $\frac{7}{8}$ " diameter A325 bolt without threads in the shear plane with reduction of a filler is:

$$R_n = 0.48 \cdot A_b \cdot F_{ub} \cdot N_s \cdot R = 0.48 \cdot 0.601 \cdot 120 \cdot 2 \cdot 0.78 = 54.0 \text{ kips}$$

$$\phi_s R_n = 0.80 \cdot 54.0 = 43.2 \text{ kips}$$

The maximum design force for Strength I,  $T_{ccfdes} = 1031.3$  kips

The number of bolts, N, required on the fill plate side of the connection is:

$$N = \frac{T_{ccfdes}}{\phi_s \cdot R_n} = \frac{1031.3}{43.2} = 23.9 \text{ bolts}$$

Use 4 rows of 6 bolts on each side of the splice.

**[6.13.2.9]**

**Bolt Bearing Resistance**

Check the bearing on the smaller flange plate (element carrying the double shear load).

$$\text{Average design force of each bolt} = \frac{1031.3}{24} = 43.0 \text{ kips}$$

Bolts will be spaced at 3 inches with an edge distance of 1.5 inches.  
Clear distance between holes =  $3 \cdot 1 = 2.0$  inches

Clear end distance =  $1.5 - 0.5 = 1.0 \text{ in} < 2.0d$

$$R_n = 1.2 \cdot L_c \cdot t \cdot F_u = 1.2 \cdot \left( 1.5 - \frac{1.0}{2} \right) \cdot 1.25 \cdot 70 = 105.0 \text{ kips}$$

$$R_r = \phi_{bb} \cdot R_n = 0.8 \cdot 105.0 = 84.0 \text{ kips} > 43.0 \text{ kips} \quad \underline{\text{OK}}$$

**[6.13.2.8]****Bolt Slip Resistance**

Slip resistance of bolts in a slip-critical connection shall be taken as:

$$R_n = K_h K_s N_s P_t$$

where:

$$K_h = 1.0 \quad K_s = 0.5 \quad N_s = 2 \quad P_t = 39.0 \text{ kips}$$

$$R_n = 1.0 \cdot 0.5 \cdot 2 \cdot 39.0 = 39.0 \text{ kips/bolt}$$

**[6.13.2.2]**

$$R_r = R_n = 39.0 \text{ kips/bolt}$$

LRFD 6.13.6.1.4a requires that connections be proportioned to prevent slip during construction as well as under service loads.

Based on Table 6.9.19, maximum design force on gross section of Service II load combination shall be taken as:

$$F_{II} = (14.70) (22) (1.25) = 404.3 \text{ kips}$$

Average design force of each bolt:

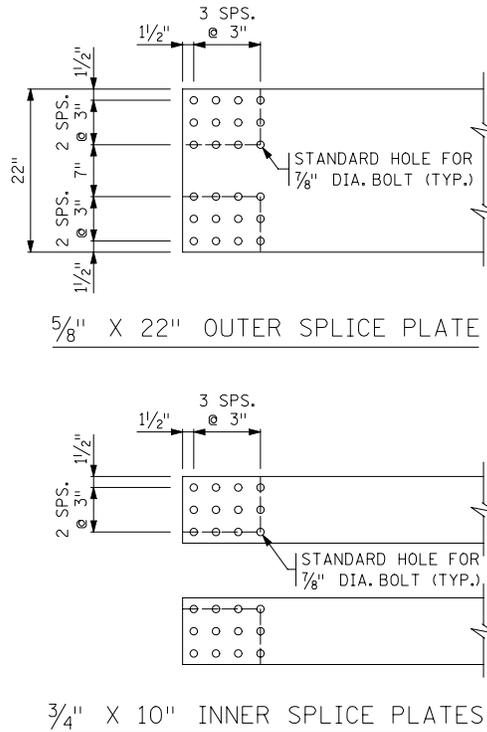
$$= \frac{404.3}{24} = 16.8 \text{ kips} < 39.0 \text{ kips} \quad \underline{\text{OK}}$$

**[6.13.4]****Block Shear Rupture Resistance**

All tension connections must be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection.

Check block shear on a transverse section through the smaller flange plate with the design force for the flange.

Possible block shear failure mode 1 on the inner and outer splice plates is shown below in Figure 6.9.10.



**Figure 6.9.10**

**Block Shear Failure Mode 1 – Bottom Flange Splice Plates**

Check Outer Splice Plate

$A_{tn}$  is the net area along the planes resisting the tensile stress:

$$A_{tn} = 2 \cdot [1.5 + 2 \cdot 3 - (2.5)(1.00)] \cdot 0.625 = 6.25 \text{ in}^2$$

$A_{vn}$  is the net area along the planes resisting the shear stress:

$$A_{vn} = 2 \cdot [1.5 + 3 \cdot 3 - 3.5(1.00)] \cdot 0.625 = 8.75 \text{ in}^2$$

$$\frac{A_{tn}}{A_{vn}} = \frac{6.25}{8.75} = 0.71 > 0.58$$

$$R_r = \phi_{bs} \cdot (0.58 \cdot F_y A_{vg} + F_u A_{tn})$$

where:

$$\phi_{bs} = 0.8$$

$A_{vg}$  is the gross area along the plane resisting shear stress:

$$A_{vg} = 2 (1.5 + 3 \cdot 3) (0.625) = 13.13 \text{ in}^2$$

$$R_r = 0.8 \cdot (0.58 \cdot 50 \cdot 13.13 + 70 \cdot 6.25) = 654.6 \text{ kips} > 442.2 \text{ kips} \quad \underline{OK}$$

Check inner splice plates

$$A_{tn} = 2 \cdot [1.5 + (2) (3) - (2.5)(1.0)] \cdot 0.75 = 7.50 \text{ in}^2$$

$$A_{vn} = 2 \cdot [1.5 + (3)(3) - 3.5(1.0)] \cdot 0.75 = 10.50 \text{ in}^2$$

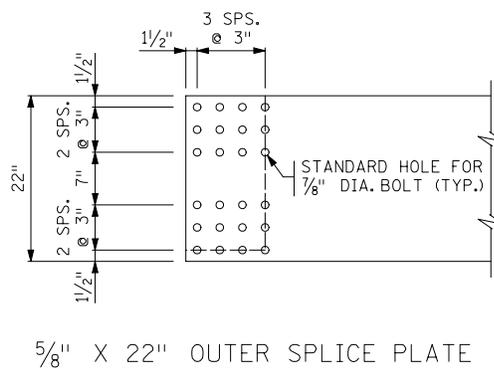
$$\frac{A_{tn}}{A_{vn}} = \frac{7.50}{10.50} = 0.71 > 0.58$$

$$A_{vg} = 2 \cdot (1.5 + 3 \cdot 3) \cdot 0.75 = 15.75 \text{ in}^2$$

$$R_r = \phi_{bs} \cdot (0.58 \cdot F_y A_{vg} + F_u A_{tn})$$

$$= 0.8 \cdot (0.58 \cdot 50 \cdot 15.75 + 70 \cdot 7.50) = 785.4 \text{ kips} > 442.2 \text{ kips} \quad \underline{OK}$$

The possible block shear failure mode 2 on the outer splice plate is shown below in Figure 6.9.11. Since the outer splice plate controlled for block shear failure mode 1, it can be seen that it will control for failure mode 2 also.



**Figure 6.9.11**

**Block Shear Failure Mode 2 - Bottom Flange Splice Plates**

For the outer splice plate,

$$A_{tn} = [1.5 + (4)(3) + (7) - (5.5)(1)] (0.625)$$

$$= 9.38 \text{ in}^2$$

$$A_{vn} = [1.5 + (3)(3) - (3.5)(1)] (0.625)$$

$$= 4.38 \text{ in}^2$$

$$\frac{A_{tn}}{A_{vn}} = \frac{9.38}{4.38} = 2.14 > 0.58$$

$$A_{vg} = (1.5 + 3 \cdot 3) (0.625) = 6.56 \text{ in}^2$$

$$R_r = \phi_{bs} (0.58 F_y A_{vg} + F_u A_{tn})$$

$$= (0.8) [(0.58) (50.0) (6.56) + (70) (9.38)] = 677.5 \text{ kips} > 442.2 \text{ kips} \quad \underline{OK}$$

**M.2 Noncontrolling  
Flange**  
**[6.13.6.1.4c]**

Table 6.9.19 indicates that the top flange is the noncontrolling flange for both positive and negative live load for the Strength I Limit State.

The noncontrolling flange at the strength limit state shall be proportioned to provide a minimum design resistance for both positive and negative live load moments equal to the greater of:

$$F_{ncf} = R_{cf} \left| \frac{f_{ncf}}{R_h} \right| \quad \text{or} \quad 0.75 \alpha \phi F_{yf}$$

where:

$$R_{cf} = \left| \frac{F_{cf}}{f_{cf}} \right|$$

Load Case 1 - Positive Live Load for Strength I Limit State  
Top flange is in compression  
From Table 6.9.19:

$$f_{cf} = 15.96 \text{ ksi} \quad f_{ncf} = -3.25 \text{ ksi} \quad F_{cf} = 37.5 \text{ ksi}$$

$$R_{cf} = \left| \frac{F_{cf}}{f_{cf}} \right| = \frac{37.5}{15.96} = 2.35$$

$$F_{ncf} = (2.35) \cdot \left| \frac{-3.25}{1.0} \right| = 7.64 \text{ ksi}$$

or

$$F_{ncf} = (0.75) (1.0) (1.0) (50) = 37.5 \text{ ksi}$$

GOVERNS

Maximum compression design force in top flange is:

$$\begin{aligned} T_{\text{cnfdes}} &= F_{\text{ncf}} \cdot A_e, \text{ where } A_e = A_g \text{ for compression flange} \\ &= (37.5) (20) (1.0) = 750.0 \text{ kips} \end{aligned}$$

Load Case 2 - Negative Live Load for Strength I Limit State

Top flange is in tension.

From Table 6.9.19:

$$f_{\text{cf}} = -19.67 \text{ ksi}$$

$$f_{\text{ncf}} = 17.39 \text{ ksi}$$

$$F_{\text{cf}} = 37.5 \text{ ksi}$$

$$R_{\text{cf}} = \left| \frac{F_{\text{cf}}}{f_{\text{cf}}} \right| = \frac{37.5}{19.67} = 1.91$$

$$F_{\text{ncf}} = R_{\text{cf}} \left| \frac{f_{\text{ncf}}}{R_n} \right| = (1.91) \left| \frac{17.39}{1.0} \right| = 33.20 \text{ ksi}$$

or

$$F_{\text{ncf}} = (0.75) (1.0) (1.0) (50) = 37.5 \text{ ksi}$$

GOVERNS

Effective area of top flange:

$$\begin{aligned} A_e &= \left( \frac{\phi_u F_u}{\phi_y F_{yt}} \right) A_n \\ &= \left( \frac{(0.8) (70)}{(0.95) (50)} \right) (1.0) (20 - (4)(1.0)) \\ &= 18.86 \text{ in}^2 < A_g = 20 \text{ in}^2 \end{aligned}$$

Maximum tension design force of top flange at splice location:

$$T_{\text{tnfdes}} = (37.5) (18.86) = 707.3 \text{ kips}$$

The design of the top flange splice is not included in this design example for brevity. However, the top flange splice is designed using the same procedures and methods presented in this example for the bottom flange splice. The size of the resulting top flange splice plates are as follows.

The outer plate is  $1/2$ " x 20" (area = 10.00 in<sup>2</sup>) and the inner plates are  $5/8$ " x 9" (area = 5.625 in<sup>2</sup> per plate).

### M.3 Web Splice

#### [6.13.6.1.4b]

The web is designed to carry the entire factored vertical shear force. In addition, it must carry the moment due to the eccentricity of the shear force and the flexural moment which the web was assumed to carry. The flexural stresses in the web are resolved into flexural and axial (horizontal) components about mid-depth of the web. This allows the bolt group on each side of the splice to be designed for the vertical shear, the moment associated with the eccentricity of the vertical shear, the web flexural moment, and the resultant horizontal force in the web.

In this example,  $M_{uw}$  and  $H_{uw}$  are computed by conservatively using the stresses at the midthickness of the flanges. By utilizing the stresses at the midthickness of the flanges, the same stress values can be used for the design of both the flange and web splices, which simplifies the calculations. The design forces will be computed under the Strength I Limit State and Service II Limit State.

Strength I Limit State:

From Tables 6.9.9 and 6.9.11, the vertical shear force to be carried is:

For positive live load shear:

$$V_u = |0.9 \cdot [(-116) + (-22)] + 1.75 \cdot 1.112(20)| = 85.3 \text{ kips}$$

For negative live load shear:

$$V_u = |(1.25) [(-116) + (-22)] + 1.75 \cdot 1.112(-114)| = 394.3 \text{ kips} \quad \underline{\text{GOVERNS}}$$

The nominal shear resistance of the unstiffened web,  $V_n$ , is 460.6 kips.

Then

$$0.5\phi_v V_n = 230.3 \text{ kips} < 394.3 \text{ kips}$$

Therefore, the design shear force is taken as:

$$\begin{aligned} V_{uw} &= \frac{(V_u + \phi_v V_n)}{2} \text{ kips} \\ &= \frac{(394.3 + (460.6)(1.0))}{2} = 427.5 \text{ kips} \end{aligned}$$

Next, determine the design moment and the design horizontal force resultant.

Load Case 1 - Strength I Limit State With Positive Live Load

[C6.13.6.1.4b]

$$M_{uw} = \frac{t_w D^2}{12} \left| R_h F_{cf} - R_{cf} f_{ncf} \right|$$

where:

$$t_w = 0.625 \text{ in}$$

$$D = 70 \text{ in}$$

$$R_h = 1.0$$

$$F_{cf} = 37.5 \text{ ksi}$$

$$R_{cf} = 2.35$$

$$f_{ncf} = -3.25 \text{ ksi}$$

$$\begin{aligned} M_{uw} &= \frac{(0.625)(70)^2}{12} \left| (1.0)(37.5) - (2.35)(-3.25) \right| \left( \frac{1}{12} \right) \\ &= 960.0 \text{ k-ft} \end{aligned}$$

$$\begin{aligned} H_{uw} &= \frac{t_w D}{2} (R_h F_{cf} + R_{cf} f_{ncf}) \\ &= \frac{(0.625)(70)}{2} [(1.0)(37.5) + (2.35)(-3.25)] \\ &= 653.2 \text{ kips} \end{aligned}$$

Assume a horizontal bolt pitch of 3 inches and two vertical rows of bolts on each side of the splice. The eccentricity of the shear is the distance from the center of the bolt pattern to the center of the splice:

$$e_v = \frac{3}{2} + \frac{3.5}{2} = 3.25 \text{ in}$$

The moment associated with the vertical shear is:

$$M_v = e_v \cdot V_{uw} = 3.25 \cdot 427.5 \left( \frac{1}{12} \right) = 115.8 \text{ k-ft}$$

Total design moment:

$$M_{uw\_pos} = 960.0 + 115.8 = 1075.8 \text{ k-ft}$$

The design forces for the web splice under positive live load condition are:

$$\begin{aligned}V_{uw} &= 427.5 \text{ kips} \\H_{uw} &= 653.2 \text{ kips} \\M_{uw\_pos} &= 1075.8 \text{ k-ft}\end{aligned}$$

Load Case 2 - Strength I Limit State With Negative Live Load

$$\begin{aligned}t_w &= 0.625 \text{ in} \\D &= 70 \text{ in} \\R_h &= 1.0 \\F_{cf} &= -37.5 \text{ ksi} \quad (\text{compression}) \\R_{cf} &= 1.91 \\f_{ncf} &= 17.39 \text{ ksi} \\M_{uw} &= \frac{t_w D^2}{12} |R_h F_{cf} - R_{cf} f_{ncf}| \\&= \frac{(0.625)(70)^2}{12} |(1.0)(-37.5) - (1.91)(17.39)| \left(\frac{1}{12}\right) \\&= 1503.9 \text{ k-ft} \\H_{uw} &= \frac{t_w D}{2} (R_h F_{cf} + R_{cf} f_{ncf}) \\&= \frac{(0.625)(70)}{2} [(1.0)(-37.5) + (1.91)(17.39)] \\&= -93.7 \text{ kips} \\M_v &= 115.8 \text{ k-ft}\end{aligned}$$

Total design moment

$$M_{uw-neg} = 1503.9 + 115.8 = 1619.7 \text{ k-ft}$$

The design forces for the web splice under negative live load condition are:

$$\begin{aligned}V_{uw} &= 427.5 \text{ kips} \\H_{uw} &= -93.7 \text{ kips} \\M_{uw-neg} &= 1619.7 \text{ k-ft}\end{aligned}$$

Load Case 3 - Service II Limit State With Positive Live Load

From Tables 6.9.9 and 6.9.11, the factored shear with positive live load is:

$$V_{\text{pos}} = (1.0) [ (-116) + (-22) ] + (1.3) (1.112) (20) = -109.1 \text{ kips}$$

Determine the design moment and the design horizontal force resultant.

From Table 6.9.19:

**[C6.13.6.1.4b]**

$$f_s = 11.88 \text{ ksi}$$

$$f_{os} = -2.43 \text{ ksi}$$

$$M_{\text{ser-w}} = \frac{t D^2}{12} \left| 1.0 f_s - 1.0 f_{os} \right|$$

$$= \frac{(0.625)(70)^2}{12} \left| 1 \cdot (11.88) - 1 \cdot (-2.43) \right| \left( \frac{1}{12} \right) = 304.3 \text{ k-ft}$$

$$H_{\text{ser-w}} = \frac{t D}{2} (1.0 f_s + 1.0 f_{os})$$

$$= \frac{(0.625)(70)}{2} [ 1 \cdot (11.88) + 1 \cdot (-2.43) ] = 206.7 \text{ k}$$

The moment from eccentricity of the shear:

$$M_v = V_{\text{ser-w}} \cdot e_v = (109.1) (3.25) \left( \frac{1}{12} \right) = 29.5 \text{ k-ft}$$

Total design moment:

$$M_{\text{ser-w}} = 304.3 + 29.5 = 333.8 \text{ k-ft}$$

The design force for the web splice under Service II Limit State with Positive Live Load is:

$$V_{\text{ser-w}} = 109.1 \text{ kips}$$

$$H_{\text{ser-w}} = 206.7 \text{ kips} \quad (\text{compression})$$

$$M_{\text{ser-w}} = 333.8 \text{ k-ft}$$

The design force values for the other Service II load cases are shown in Table 6.9.20.

## Load Case 4 - Fatigue

The fatigue detail category for a bolted connection is Category B. The splice is to be designed for infinite fatigue life and the Category B constant amplitude fatigue threshold  $(\Delta F)_{TH}$  is 16 ksi.

$$\text{Then } \Delta F_n = 0.5 \cdot (\Delta F)_{TH} = 0.5 \cdot 16 = 8.0 \text{ ksi}$$

$$\text{Max Actual range } \gamma \cdot (\Delta f) = 3.29 \text{ ksi} < 8.0 \text{ ksi} \quad \underline{\text{OK}}$$

**Bolt Shear Resistance**

The vertical shear and the horizontal force are assumed to be resisted equally by all bolts in the fastener group. The force carried by each of the bolts to resist flexure is assumed to be proportional to its distance from the center of the fastener group.

The force in each of the bolts can be found with the following equations:

$$R_{xA} = R_{xp} + R_{xm} = \frac{P_x}{nm} + \frac{M \cdot y_A}{I_p} \qquad R_{yA} = R_{yp} + R_{ym} = \frac{P_y}{nm} + \frac{M \cdot x_A}{I_p}$$

$$R_A = \sqrt{R_{xA}^2 + R_{yA}^2}$$

$$I_p = \frac{nm}{12} [s^2(n^2 - 1) + g^2(m^2 - 1)]$$

where:

$$P_x = H_{uw} \qquad P_y = V_{uw} \qquad M = M_{uw}$$

$x_A$  = x coordinate of bolt

$y_A$  = y coordinate of bolt

$I_p$  = polar moment of inertia of the bolt group

$n$  = number of bolts per row

$m$  = number of vertical rows of bolts

$s$  = vertical pitch

$g$  = horizontal pitch

**[C6.13.6.1.4b]**

Assume two vertical rows of 22 bolts on each side of the splice, a horizontal pitch  $g$  of 3 inches and a vertical pitch  $s$  of 3 inches. The bolts at the corners of the fastener group will be subject to the largest forces. Conservatively, the corner bolts will be checked only. The coordinates at the corners are  $x = \pm 1\frac{1}{2}$  and  $y = \pm 31.5$  inches.

Substituting values in the above equations, Table 6.19.20 shows the design force in the corner bolts under the Strength I Limit State and Service II Limit State.

**Table 6.9.20**  
**Design Force of the Corner Bolts**

Parameter	Strength I		Service II		Service II Const.	
	M(+)	M(-)	M(+)	M(-)	M(+)	M(-)
M <sub>uw</sub> (k-ft)	1075.8	1619.7	333.9	663.9	710.5	684.4
H <sub>uw</sub> (kip) *	653.2	-93.7	206.7	-36.3	-57.3	56.4
V <sub>uw</sub> (kip)	427.5	427.5	109.1	302.8	109.0	52.0
n	22	22	22	22	22	22
m	2	2	2	2	2	2
s (in)	3	3	3	3	3	3
g (in)	3	3	3	3	3	3
y (in)	31.5	-31.5	1.5	1.5	1.5	1.5
x (in)	1.5	1.5	31.5	-31.5	-31.5	31.5
I <sub>p</sub> (in <sup>2</sup> )	16038	16038	16038	16038	16038	16038
R <sub>x</sub> (kip)	40.2	-40.3	12.6	-16.5	-18.0	17.4
R <sub>y</sub> (kip)	10.9	11.5	2.9	7.6	3.3	1.9
R (kip)	41.7	41.9	12.9	18.2	18.3	17.5

\* H<sub>uw</sub> is a signed quantity, positive for tension and negative for compression.

The nominal shear resistance of a 7/8" diameter A325 bolt without threads in the shear plane and without fill plates used is:

[6.13.2.7]

$$\begin{aligned}
 R_n &= 0.48 A_b F_{ub} N_s \\
 &= (0.48) (0.601) (120) (2) \\
 &= 69.2 \text{ kips}
 \end{aligned}$$

$$\begin{aligned}
 R_r &= \phi R_n \\
 &= (0.8) (69.2) \\
 &= 55.4 \text{ kips}
 \end{aligned}$$

From Table 6.9.20, the maximum design force on the bolt at Strength I Limit State

$$R_{STRI} = 41.9 \text{ kips} < 55.4 \text{ kips} \quad \text{OK}$$

**Bolt Slip Resistance**

The nominal slip resistance of a  $7/8$ " diameter A325 bolt is:

**[6.13.2.8]**

$$\begin{aligned} R_n &= K_h K_s N_s P_t \\ &= (1.0) (0.5) (2) (39.0) \\ &= 39.0 \text{ kips} \\ R_r &= R_n = 39.0 \text{ kips} \end{aligned}$$

From Table 6.9.20, the maximum design force on the bolt at Service II Limit State is:

$$R_{sII} = 18.3 \text{ kips} < 39.0 \text{ kips} \quad \text{OK}$$

**Bolt Bearing Resistance**

Nominal bearing resistance of interior or end bolt holes at the Strength I Limit State depends on the clear distance between the holes or clear end distance.

$$\text{Clear distance between holes} = 3 - 1 = 2" > 2d = 1.75"$$

$$\text{Clear end distance} = 1.75 - 0.5 = 1.25" < 2d = 1.75"$$

**[6.13.2.9-2]**

$$\begin{aligned} R_n &= 1.2 L_c t F_u \\ L_{c1} &= 3.0 - 1.0 = 2.0" && \text{for interior bolts} \\ L_{c2} &= 1.75 - 0.5 = 1.25" && \text{for end bolts} \end{aligned}$$

$$R_{n\_int} = (1.2) (2.0) (0.625) (70) = 105 \text{ kips}$$

$$R_{n\_end} = (1.2) (1.25) (0.625) (70) = 65.6 \text{ kips} \quad \text{GOVERNS}$$

$$\begin{aligned} R_r &= \phi_{bb} R_{n\_end} \\ &= (0.8) (65.6) \\ &= 52.5 \text{ kips} \end{aligned}$$

From Table 6.9.20, maximum design force at Strength I Limit State

$$R_{STRI} = 41.9 \text{ kips} < 52.5 \text{ kips} \quad \text{OK}$$

The plates used in the web splice must have adequate resistance to carry the vertical shear. Two  $3/8$ " thick plates are being used for the splice. Assume the plates are 66 inches tall ( $21 \cdot 3 + 2 \cdot 1\frac{1}{2}$ )

Gross area of the plates:

$$A_g = 66 \cdot 2 \cdot 0.375 = 49.50 \text{ in}^2$$

Assumed vertical shear resistance:

[6.13.5.3-2]

$$R_n = 0.58 \cdot F_y \cdot A_g = 0.58 \cdot 50 \cdot 49.50 = 1435.5 \text{ kips}$$

[6.13.5.3-1]

$$R_r = \phi_v R_n = (1.0) (1435.5) = 1435.5 \text{ kips} > V_{uw} = 427.5 \text{ kips} \quad \underline{\text{OK}}$$

### Block Shear Rupture Resistance

[6.13.4]

Check block shear failure mode shown in Figure 6.9.12 for web splice plates.

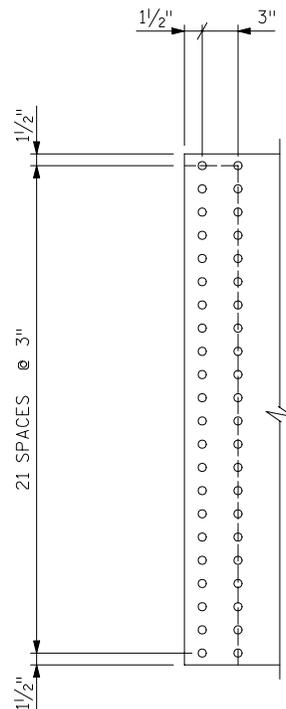


Figure 6.9.12

Net area along the plane resisting shear:

$$A_{vn} = (2) (66 - 1.5 - (21.5) (1.0)) (0.375) = 32.25 \text{ in}^2$$

Net area along the plane resisting tension:

$$A_{tn} = (2) (1.5 + 3.0 - (1.5) (1.0)) (0.375) = 2.25 \text{ in}^2$$

$$\frac{A_{tn}}{A_{vn}} = \frac{2.25}{32.25} = 0.07 < 0.58$$

$$R_r = \phi_{bs} (0.58 F_u A_{vn} + F_y A_{tg})$$

$$A_{tg} = (2) (1.5 + 3.0) (0.375) = 3.38 \text{ in}^2$$

$$R_r = (0.8) [ (0.58) (70) (32.25) + (50) (3.38) ]$$

$$= 1182.7 \text{ kips} > V_{uw} = 427.5 \text{ kips} \quad \text{OK}$$

At Strength I Limit State the flexure stress in the web splice plates shall not exceed  $\phi_f F_y$ :

[6.13.6.1.4b]

$$\sigma = \frac{H_{uw}}{A_g} + \frac{M_{uw}}{S_{pl}} \leq \phi_f F_y$$

Section modulus of the web splice plates is:

$$S_{pl} = 2 \cdot \frac{t_w \cdot D_{sp}^3}{12} \cdot \frac{1}{D_{sp}/2} = \frac{t_w \cdot D_{sp}^2}{3} = \frac{0.375 \cdot 66^2}{3} = 544.5 \text{ in}^3$$

$$\phi_f = 1.0$$

For positive liveload moment of Strength I limit State:

$$H_{uw} = 653.2 \text{ kips}$$

$$M_{uw} = 1075.8 \text{ k-ft}$$

$$\sigma = \frac{653.2}{49.50} + \frac{(1075.8)(12)}{544.5} = 36.9 \text{ ksi} < 50 \text{ ksi} \quad \text{OK}$$

For negative liveload moment of Strength I limit State:

$$H_{uw} = -93.7 \text{ kips}$$

$$M_{uw} = 1619.7 \text{ k-ft}$$

$$\sigma = \frac{93.7}{49.50} + \frac{(1619.7)(12)}{544.5} = 37.6 \text{ ksi} < 50 \text{ ksi} \quad \text{OK}$$

The assumed web splice details have adequate capacity. The field splice is detailed in Figure 6.9.13.

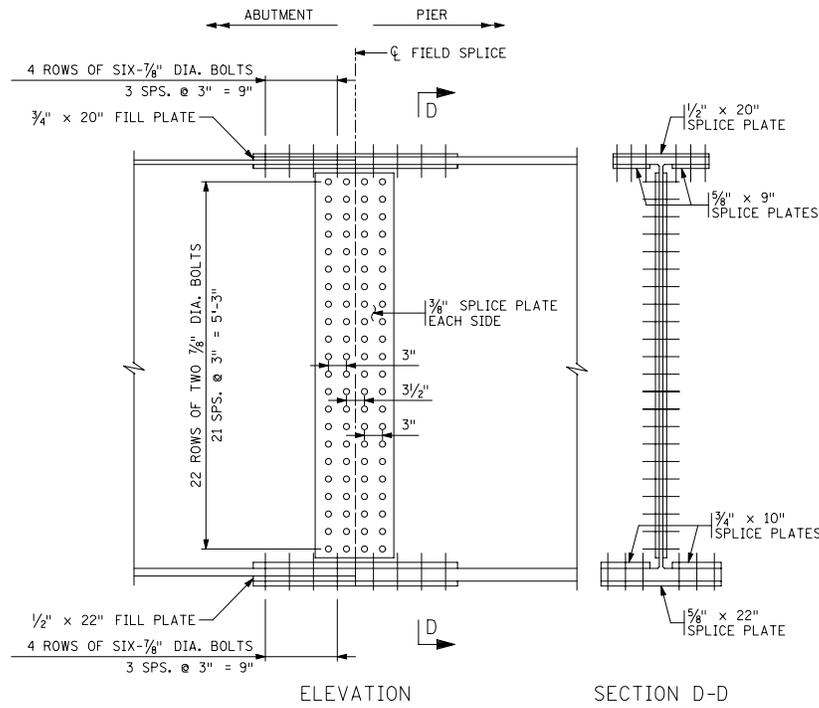


Figure 6.9.13

**N. Investigate Deflection**

Consider with the Service I load combination. No sidewalk or bicycle path is provided on the bridge. In accordance with Mn/DOT policy, the live load deflection limit is  $L / 800$ . The maximum deflection permitted for this example is:

$$L / 800 = 175 \cdot (12) / 800 = 2.63 \text{ in}$$

Two live loads are applied to the bridge and evaluated for the deflection check. Take the larger of:

**[3.6.1.3.2]**

- Design Truck alone
- 25% of Design Truck + Lane Loading

When computing deflections a separate distribution factor is used. It is simply the number of design lanes divided by the number of girders. Mn/DOT practice is to use a multiple presence factor for deflections of no less than 0.85 (See Section 3.4.2). For this example, the distribution factor is:

**[2.5.2.6.2]**

$$g_{\Delta} = \frac{\text{Number of Design Lanes}}{\text{Number of Girders}} \cdot \Delta_{MPF} = \frac{4}{5} \cdot 0.85 = 0.68$$

The maximum deflections (like the moments) are based on the composite section, including the deck in the negative regions. Including dynamic load allowance, the maximum deflections for a full lane or truck are:

$$\Delta_{\text{max}+l} = 1.45 \text{ inches at } 0.45 \text{ of span} < 2.63 \text{ in} \quad \text{OK}$$

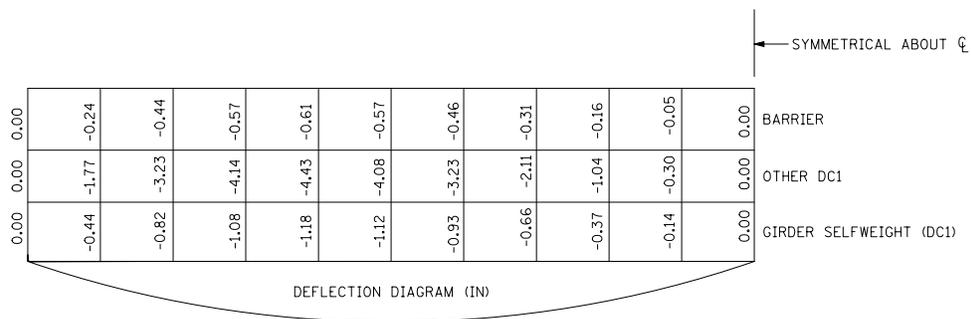
**O. Camber**

To ensure that steel bridges have the proper profile after construction, steel girders are fabricated with camber. Camber is an adjustment to the vertical profile of a girder. Camber in the girder is made up of geometric camber, dead load camber, and residual camber (if required).

This bridge is on a straight grade, so it will not require any geometric camber.

The girders for this example will deflect 1.18 inches downward at the 0.4 Span Point due to their own weight. When the other DC1 dead loads (deck, stool) are added to the bridge, an additional 5.61 inches of downward deflection is estimated for the 0.4 Span Point. The addition of barriers will add an additional deflection of 0.61 inches downward at the same location. Summing these values results in an anticipated deflection of 7.40 inches. Deflections at 10<sup>th</sup> points along the span are provided for selfweight, other DC1 loads, and DC2 loads in Figure 6.9.14.

As previously calculated, a residual camber of 2.5 inches is also provided. The residual camber is provided to prevent the appearance of a sag in the span.



**Figure 6.9.14**

By following the procedure in 6.3.4, the camber diagram is developed and shown on Figure 6.9.15.

Figure 6.9.16 contains a half elevation of the girder that summarizes the design.

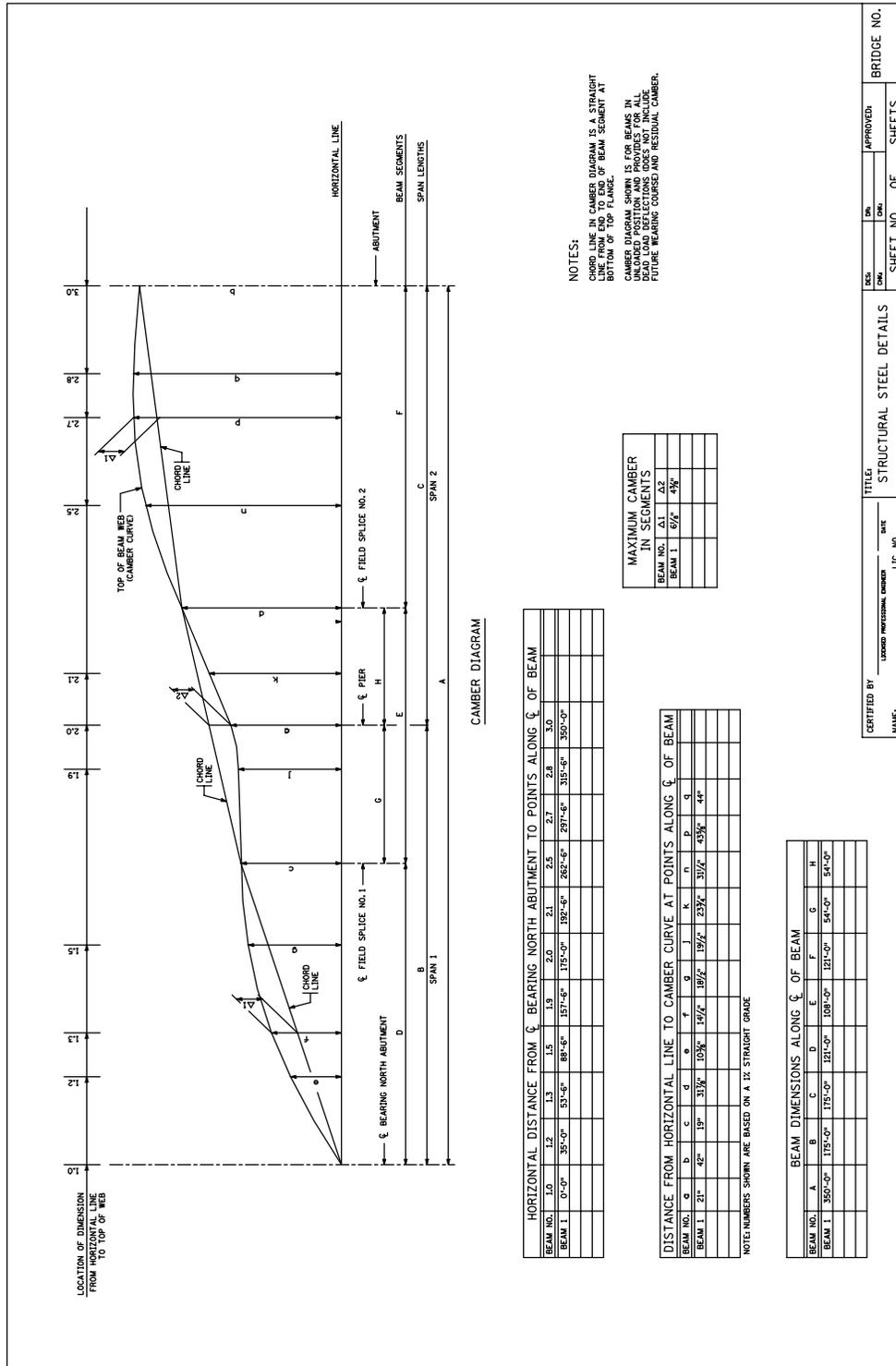


Figure 6.9.15

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 LIC. NO.: \_\_\_\_\_  
 TITLE: STRUCTURAL STEEL DETAILS  
 SHEET NO. ... OF ... SHEETS  
 BRIDGE NO. \_\_\_\_\_

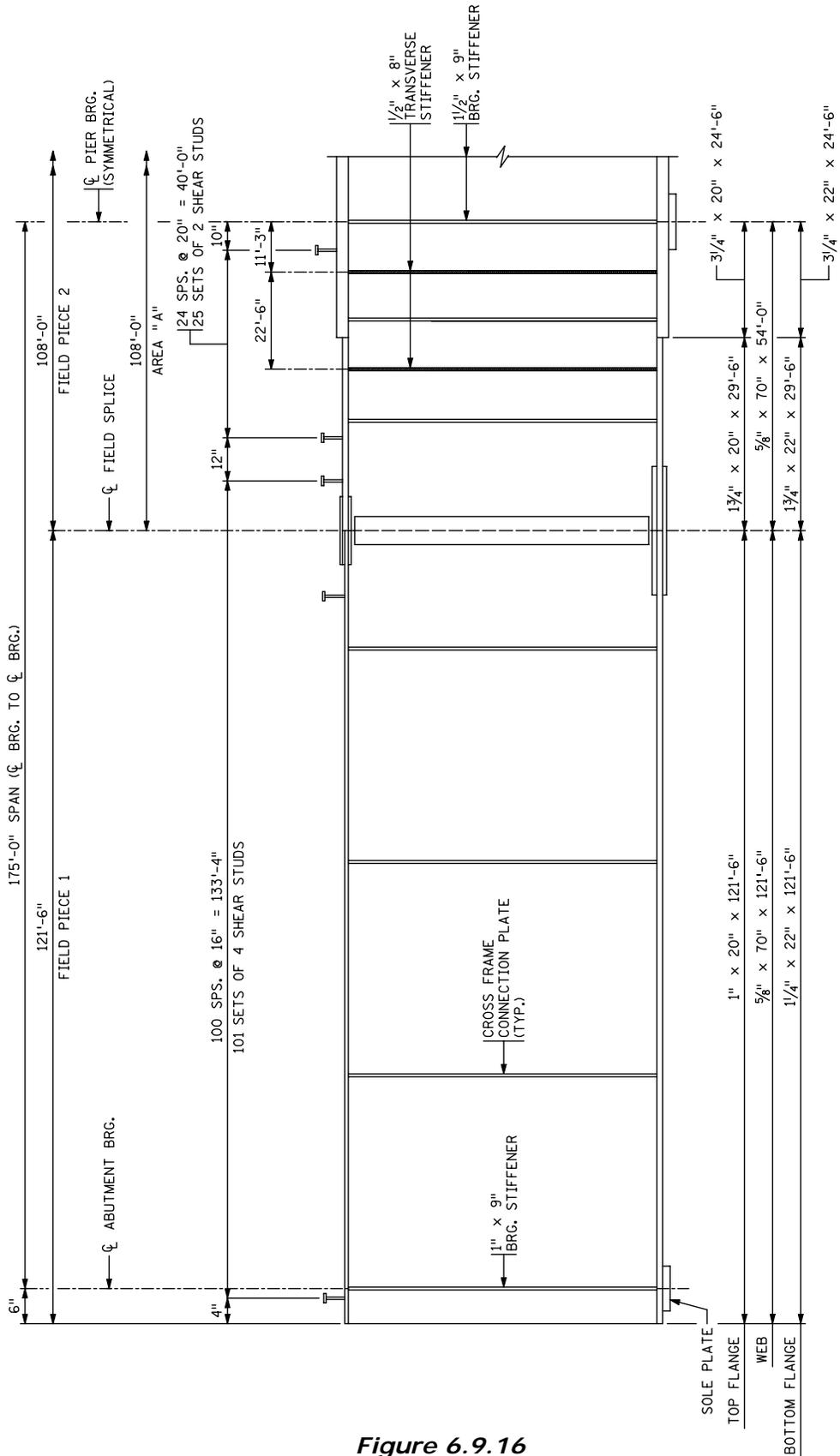
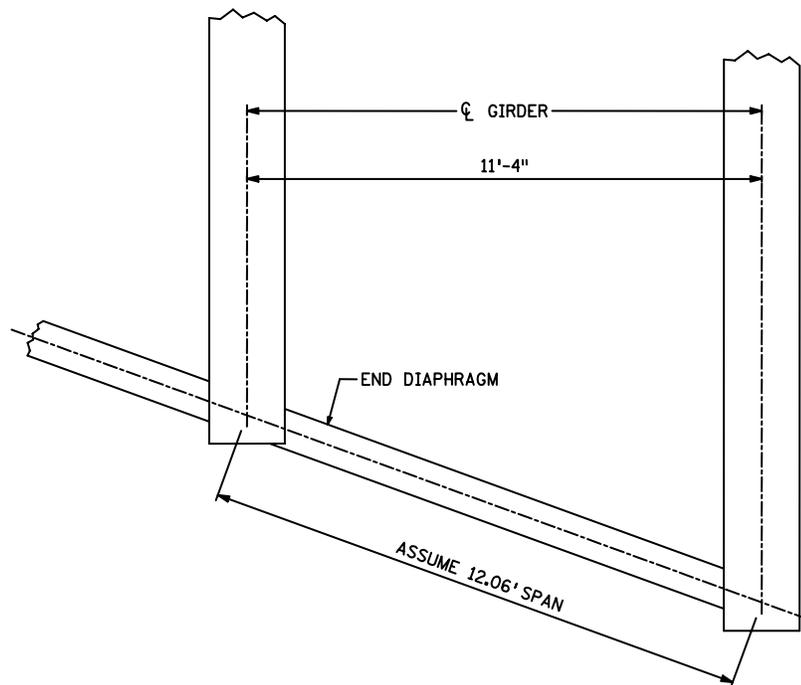


Figure 6.9.16  
Half Elevation

**P. End Diaphragm Design**

The end diaphragm is used to support the end of the deck and to transfer wind load to the supports. It also is required to carry jacking loads if the bearings are replaced. Compared to the jacking loads and the dead and live loads, the wind loads for this example are relatively modest. The end diaphragm will be designed for two load combinations: Strength I where dead and live loads are carried on a simple non-composite span, and Strength I where dead loads and jacking loads are carried on simple span as well.

The design simple span length will be the distance between girders increased for the skew. See Figure 6.9.16.



**Figure 6.9.17**  
**Length of End Diaphragm**

Assume that the end diaphragm carries its own selfweight, the weight of a 2 foot strip of deck, and the additional weight of the thickened deck at the joint. For dead load purposes, assume the additional thickness is 4 inches and that it is 14 inches wide.

Assume 50 pounds per lineal foot for the weight of the beam and steel connections.

The assumed dead load per foot is:

$$w_d = 0.050 + 0.150 \cdot \left( 2 \cdot \frac{9.5}{12} + \frac{14}{12} \cdot \frac{4}{12} \right) = 0.346 \text{ kips/ft}$$

Dead load shear is:

$$\frac{w_d \cdot L}{2} = \frac{0.346 \cdot 12.06}{2} = 2.1 \text{ kips}$$

Dead load moment is

$$\frac{w_d \cdot L^2}{8} = \frac{0.346 \cdot 12.06^2}{8} = 6.3 \text{ kips-ft}$$

Consider two live load cases, one where the lane of traffic is centered between the girders and a second one where one of the truck wheels is placed at the center of the diaphragm. The two cases are presented in Figure 6.9.18. This assumes two feet of lane load and includes dynamic load allowance on the wheel load. For Case 1, the live load is centered between the girders and the shear force is:

$$V = 21.3 + 0.064 \cdot 2 \cdot \frac{10}{2} = 21.9 \text{ kips}$$

The moment at mid span for this case is:

$$M \approx 21.3 \cdot 2.67 + \frac{0.064 \cdot 2 \cdot 12.06^2}{8} = 59.2 \text{ k-ft}$$

For Case 2, assume that the left wheel is just to the right of the interior girder. This will produce a conservative design shear. The shear force for this case is:

$$V = 21.3 + \frac{6.06}{12.06} \cdot 21.3 + \frac{8.06}{12.06} \cdot [8 \cdot 2 \cdot 0.064] = 32.7 \text{ kips}$$

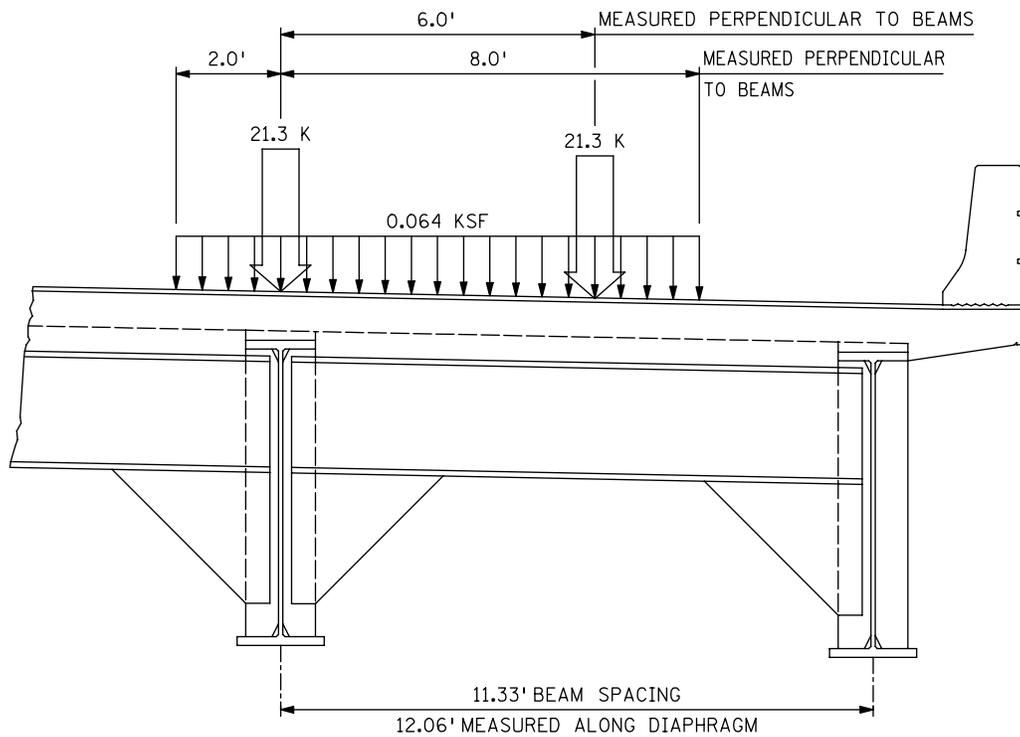
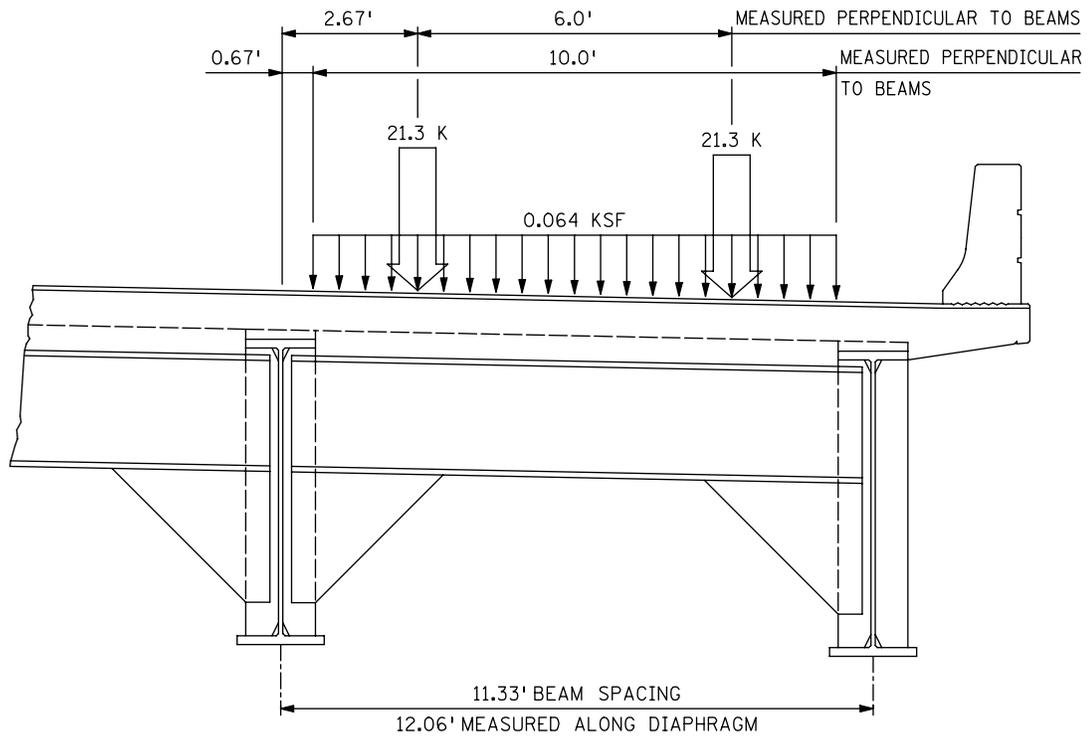
The moment at mid span for this case is:

$$\begin{aligned} M &\approx \frac{6.06}{12.06} \cdot 21.3 \cdot 6.03 + \frac{8.06}{12.06} \cdot [0.064 \cdot 2 \cdot 8] \cdot 6.03 - [0.064 \cdot 2] \cdot \frac{(6.03)^2}{2} \\ &= 71.0 \text{ k - ft} \end{aligned}$$

**[3.4.3.1]**

Assuming lane closed during jacking operation, the shear force in the end diaphragms during jacking can be estimated from the abutment reactions for the DC1 and DC2 loads. Jacking forces have a 1.3 load factor. Assume that two jacks are used to lift each interior girder and that they are placed two feet away from the center of the girder to clear the bearings.

$$1.3V_{\text{jack}} = \frac{1.3 \cdot (\text{DC1} + \text{DC2})}{2} = \frac{1.3 \cdot (115 + 23)}{2} = 89.7 \text{ kips}$$



**Figure 6.9.18**  
**Live Load Placement on End Diaphragm**

With each jack positioned two feet from the girder the moment at mid span in the end diaphragm is:

$$1.3M_{\text{jack}} = 89.7 \cdot 2 = 179.4 \text{ k-ft}$$

**[6.10.9.2-2]**

By inspection, the jacking operation governs the design of the end diaphragm. Begin by sizing a rolled beam based on shear capacity. Assume the rolled section will satisfy the slenderness ratio to permit that  $V_p$  equals 58% of the yield stress to be used.

$$\begin{aligned} V_r = V_u &= 1.25 V_{DL} + 1.3 V_{\text{jack}} \\ &= (1.25)(2.1) + 89.7 \\ &= 92.3 \text{ kips} \end{aligned}$$

$$V_r = \phi_v V_n = \phi_v C V_p$$

where:

$$\phi_v = 1.0$$

$$C = 1.0$$

Therefore,

$$V_r = (1.0)(1.0)(0.58)F_y A_w$$

$$A_w = \frac{V_r}{0.58 F_y} = \frac{92.3}{0.58 \cdot 50} = 3.18 \text{ in}^2 \text{ web area required}$$

Assume that the rolled beam can reach  $M_y$ . The required section modulus for the beam is:

$$S_{\text{required}} = \frac{(1.25)M_{DL} + (1.3)M_{\text{jack}}}{F_y} = \frac{(1.25 \cdot 6.3 + 179.4) \cdot 12}{50} = 44.9 \text{ in}^3$$

Based on review of section properties in the *AISC Manual of Steel Construction*, try a W12x40 section:

Area	A	=	11.8 in <sup>2</sup>
Depth	D	=	12.0 in
Web Thickness	t <sub>w</sub>	=	<sup>5</sup> / <sub>16</sub> in
Flange Width	b <sub>f</sub>	=	8 in
Flange Thickness	t <sub>f</sub>	=	<sup>1</sup> / <sub>2</sub> in
Section Modulus	S	=	51.9 in <sup>3</sup>
Radius of Gyration	r <sub>t</sub>	=	1.9 in

Determine the nominal shear capacity with LRFD Article 6.10.9.2.

$$\frac{D}{t_w} = \frac{12 - 2 \cdot (0.5)}{0.31} = 35.5$$

$$1.12 \cdot \sqrt{\frac{E \cdot k}{F_y}} = 1.12 \cdot \sqrt{\frac{29,000 \cdot 5}{50}} = 60.3 > 35.5$$

Therefore, C = 1.0 and the shear capacity of the beam is:

**[6.10.9.2-1]**

$$V_n = C \cdot V_p = 1 \cdot 0.58 \cdot 50 \cdot [12 - 2 \cdot 0.5] \cdot 0.31 = 98.9 \text{ kips}$$

$$V_u = 1.3 V_{\text{jack}} + (1.25) V_{\text{DL}}$$

$$= 92.3 \text{ kips} < \phi_v V_n = (1.0)(98.9) = 98.9 \text{ kips} \quad \text{OK}$$

**[6.10.8.2]**

Check Compression flange flexural resistance

Local Buckling

$$\lambda_f = \frac{b_f}{2 \cdot t_f} = \frac{8}{2 \cdot 0.5} = 8 \leq \lambda_{pf} = 9.2$$

$$F_{nc} = F_{yc} = 50.0 \text{ ksi}$$

Section is subject to compact section.

Lateral Torsional Buckling

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = (1.0)(1.93) \left( \sqrt{\frac{29,000}{50}} \right) = 46.5 \text{ in}$$

$$L_r = \pi r_{u t} \sqrt{\frac{E}{F_{yr}}} = (3.14)(1.93) \left( \sqrt{\frac{29,000}{35}} \right) = 174.0 \text{ in}$$

The distance between gusset plates is approximately 60 inches.

$$L_b = 60 \text{ in}$$

$$L_p < L_b < L_r$$

$$F_{nc} = C_b \left[ 1 - \left( 1 - \frac{F_{yr}}{R_h F_{yc}} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] R_b R_h F_{yc}$$

$$R_h = 1.0$$

$$F_{yr} = 0.7 F_{yc} = 35 \text{ ksi}$$

$$\begin{aligned} F_{nc} &= (1.0) \left[ 1 - \left( 1 - \frac{35}{50} \right) \left( \frac{60 - 46.5}{174 - 46.5} \right) \right] (1.0) (1.0) (50) \\ &= 48.4 \text{ ksi} \end{aligned}$$

**[6.10.8.2.3-6]**

$$f_2 = 0 \quad \text{Therefore, } C_b = 1.0$$

**[6.10.1.10.2-4]**

$$\lambda_{rw} = 5.7 \sqrt{\frac{E}{F_{yc}}} = 137$$

In order to determine web load-shedding factor,  $R_b$ , the web shall satisfy:

**[6.10.1.10.2-2]**

$$\frac{2D_c}{t_w} = \frac{2(12 - (0.5)(2))}{0.31} = 71 < \lambda_{rw}$$

$$R_b = 1.0$$

Flexural Resistance

$$M_n = F_y \cdot S = (48.4) (51.9) = 2512 \text{ k-in} = 209.3 \text{ k-ft}$$

$$M_r = \phi M_n = (0.9) (209.3) = 188.4 \text{ k-ft}$$

$$M_u = 1.25 M_{DL} + 1.3 M_{jack}$$

$$= (1.25) (6.3) + 179.4$$

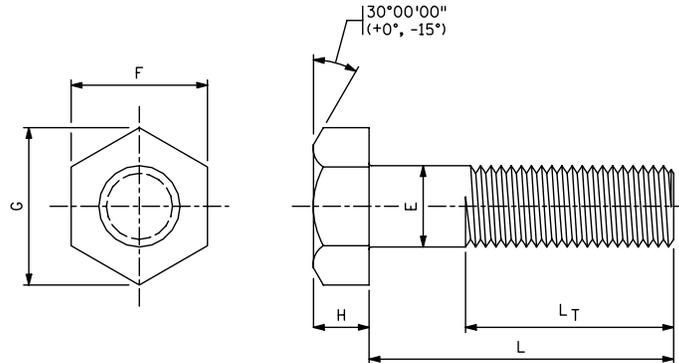
$$= 187.3 \text{ k-ft} < M_r$$

OK

Therefore, use a W12x40 for the end diaphragm.

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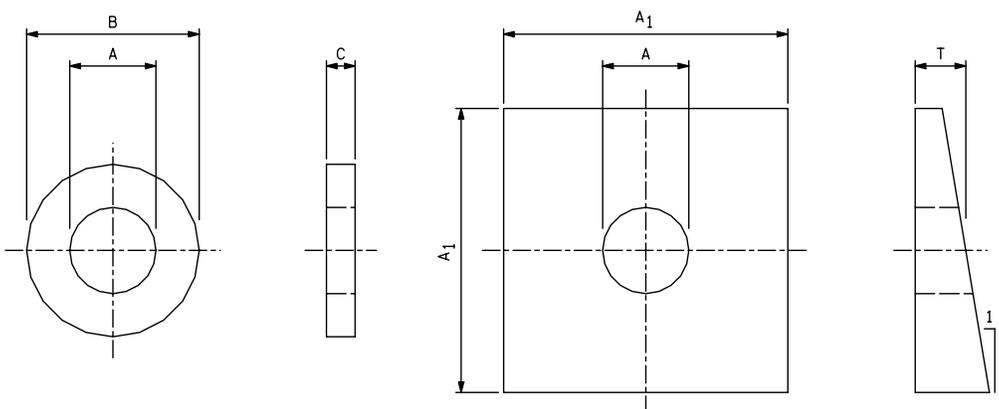
APPENDIX 6-A



NOMINAL BOLT DIAMETER	E	F				G		H			L <sub>T</sub>	
	BODY DIA.	WIDTH ACROSS FLATS				WIDTH ACROSS CORNERS		HEIGHT			THREAD LENGTH FOR BOLT LENGTHS	
		MAX.	BASIC	MAX.	MIN.	MAX.	MIN.	BASIC	MAX.	MIN.	L ≤ 6"	L > 6"
1/2	0.515	7/8	0.875	0.850	1.010	0.969	1 1/32	0.364	0.302	1.250	1.500	
5/8	0.642	1 1/16	1.062	1.031	1.227	1.175	2 1/64	0.444	0.378	1.500	1.750	
3/4	0.768	1 1/4	1.250	1.212	1.443	1.383	1/2	0.524	0.455	1.750	2.000	
7/8	0.895	1 7/16	1.438	1.394	1.660	1.589	3 7/64	0.604	0.531	2.000	2.250	
1	1.022	1 5/8	1.625	1.575	1.876	1.796	4 3/64	0.700	0.591	2.250	2.500	
1 1/8	1.149	1 13/16	1.812	1.756	2.093	2.002	3/4	0.780	0.658	2.500	2.750	
1 1/4	1.277	2	2.000	1.938	2.309	2.209	2 7/32	0.876	0.749	2.750	3.000	
1 3/8	1.404	2 3/16	2.188	2.119	2.526	2.416	2 9/32	0.940	0.810	3.000	3.250	
1 1/2	1.531	2 3/8	2.375	2.300	2.742	2.622	1	1.036	0.902	3.250	3.500	
1 3/4	1.785	2 3/4	2.750	2.662	3.175	3.035	1 1/32	1.196	1.054	3.750	4.000	
2	2.039	3 1/8	3.125	3.025	3.608	3.449	1 1/32	1.388	1.175	4.250	4.500	
2 1/4	2.305	3 1/2	3.500	3.388	4.041	3.862	1 1/2	1.548	1.327	4.750	5.000	
2 1/2	2.559	3 7/8	3.875	3.750	4.474	4.275	1 21/32	1.708	1.479	5.250	5.500	
2 3/4	2.827	4 1/4	4.250	4.112	4.907	4.688	1 13/16	1.869	1.632	5.750	6.000	
3	3.081	4 5/8	4.625	4.475	5.340	5.102	2	2.060	1.815	6.250	6.500	

**Figure 6-A1**  
**Dimensions of Common Heavy Hex Structural Bolts**  
**Mn/DOT 3391.2A (ASTM A307)**

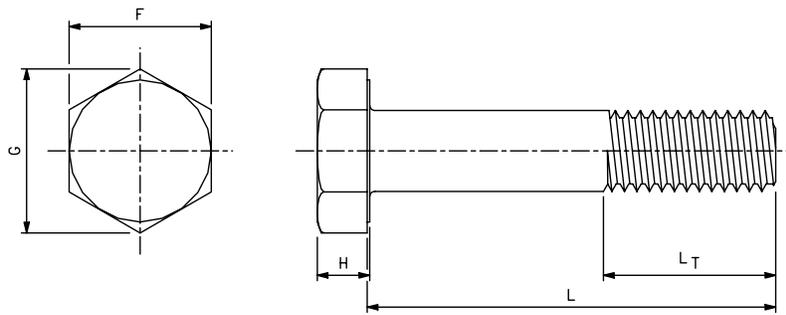
APPENDIX 6-A (Continued)



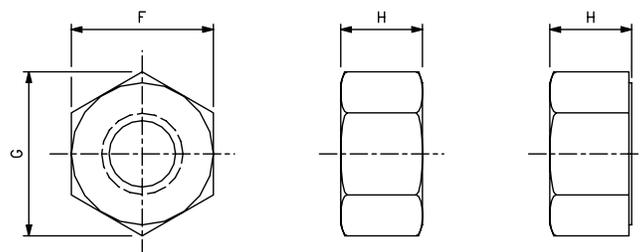
NOMINAL BOLT DIAMETER	FLAT CIRCULAR WASHERS				BEVELED WASHERS	
	A	B	C		A <sub>1</sub>	T
	INSIDE DIA.	OUTSIDE DIA.	THICKNESS		SIDE DIMENSION	MEAN THICKNESS
	NOM.	NOM.	MAX.	MIN.	NOM.	NOM.
1/2	0.531	1.062	0.177	0.097	1.750	0.312
5/8	0.688	1.312	0.177	0.122	1.750	0.312
3/4	0.812	1.469	0.177	0.122	1.750	0.312
7/8	0.938	1.750	0.177	0.136	1.750	0.312
1	1.125	2.000	0.177	0.136	1.750	0.312
1 1/8	1.250	2.250	0.177	0.136	2.250	0.312
1 1/4	1.375	2.500	0.177	0.136	2.250	0.312
1 3/8	1.500	2.750	0.177	0.136	2.250	0.312
1 1/2	1.625	3.000	0.177	0.136	2.250	0.312

**Figure 6-A2**  
**Washers for High Strength Structural Bolts**  
**Mn/DOT 3391.2B (ASTM A325)**

APPENDIX 6-A (Continued)



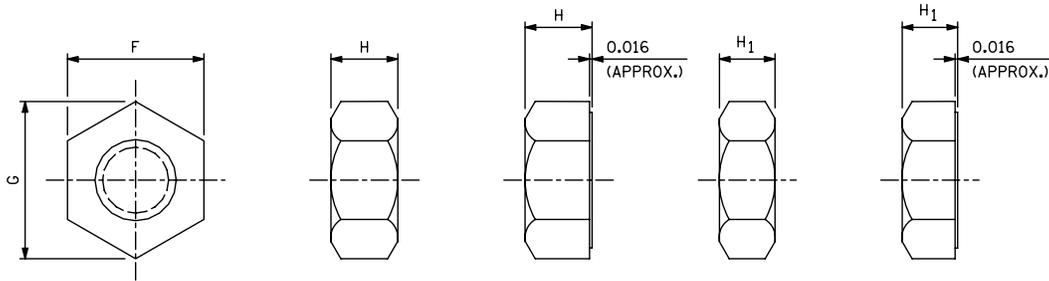
HIGH STRENGTH HEAVY HEX NUTS							
NOMINAL BOLT DIAMETER	F		G		H		L <sub>T</sub>
	WIDTH ACROSS FLATS		WIDTH ACROSS CORNERS		HEIGHT		THREAD LENGTH
	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.	BASIC
1/2	0.875	0.850	1.010	1.969	0.323	0.302	1.00
5/8	1.062	1.031	1.227	1.175	0.403	0.378	1.25
3/4	1.250	1.212	1.443	1.382	0.483	.0455	1.38
7/8	1.438	1.394	1.660	1.589	0.563	0.531	1.50
1	1.625	1.575	1.876	1.796	0.627	0.591	1.75
1 1/8	1.812	1.756	2.093	2.002	0.718	0.658	2.00
1 1/4	2.000	1.938	2.309	2.209	0.813	0.749	2.00
1 3/8	2.188	2.119	2.526	2.416	0.878	.0810	2.25
1 1/2	2.375	2.300	2.742	2.622	0.974	0.902	2.25



HIGH STRENGTH HEAVY HEX NUTS						
NOMINAL BOLT DIAMETER	F		G		H	
	WIDTH ACROSS FLATS		WIDTH ACROSS CORNERS		THICKNESS	
	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.
1/2	0.875	0.850	1.010	1.969	0.504	0.464
5/8	1.062	1.031	1.227	1.175	0.631	0.587
3/4	1.250	1.212	1.443	1.382	0.758	0.710
7/8	1.438	1.394	1.660	1.589	0.885	0.833
1	1.625	1.575	1.876	1.796	1.012	0.956
1 1/8	1.812	1.756	2.093	2.002	1.139	1.079
1 1/4	2.000	1.938	2.309	2.209	1.251	1.187
1 3/8	2.188	2.119	2.526	2.416	1.378	1.310
1 1/2	2.375	2.300	2.742	2.622	1.505	1.433

Figure 6-A3  
 High Strength Heavy Hex Structural Bolts and Nuts  
 Mn/DOT 3391.2B (ASTM A325)

APPENDIX 6-A (Continued)



NOMINAL BOLT DIAMETER	F			G		H			H <sub>1</sub>		
	WIDTH ACROSS FLATS			WIDTH ACROSS CORNERS		THICKNESS HEAVY HEX NUTS			THICKNESS HEAVY HEX JAM NUTS		
	BASIC	MAX.	MIN.	MAX.	MIN.	BASIC	MAX.	MIN.	BASIC	MAX.	MIN.
1/4	1/2	0.500	0.488	0.577	0.556	15/64	0.250	0.218	11/64	0.188	0.156
3/16	9/16	0.562	0.546	0.650	0.622	19/64	0.314	0.280	13/64	0.220	0.186
3/8	11/16	0.688	0.669	0.794	0.763	23/64	0.377	0.341	15/64	0.252	0.216
7/16	3/4	0.750	0.728	0.866	0.830	27/64	0.441	0.403	17/64	0.285	0.247
1/2	7/8	0.875	0.850	1.010	0.969	31/64	0.504	0.464	19/64	0.317	0.227
9/16	15/16	0.938	0.909	1.083	1.037	35/64	0.568	0.526	21/64	0.349	0.307
5/8	1 1/16	1.062	1.031	1.227	1.175	39/64	0.631	0.587	23/64	0.381	0.337
3/4	1 1/4	1.250	1.212	1.443	1.382	47/64	0.758	0.710	27/64	0.446	0.398
7/8	1 1/8	1.438	1.394	1.660	1.589	55/64	0.885	0.833	31/64	0.510	0.458
1	1 5/8	1.625	1.575	1.876	1.796	63/64	1.012	0.956	35/64	0.575	0.519
1 1/8	1 13/16	1.812	1.756	2.093	2.002	17/64	1.139	10.79	39/64	0.639	0.579
1 1/4	2	2.000	1.938	2.309	2.209	17/32	1.251	1.187	23/32	0.751	0.687
1 3/8	2 3/16	2.188	2.119	2.526	2.416	11/32	1.378	1.310	25/32	0.815	0.747
1 1/2	2 3/8	2.375	2.300	2.742	2.622	15/32	1.505	1.433	27/32	0.880	0.808
1 5/8	2 9/16	2.562	2.481	2.959	2.828	19/32	1.632	1.556	29/32	0.944	0.868
1 3/4	2 3/4	2.750	2.662	3.175	3.035	123/32	1.759	1.679	31/32	1.009	0.929
1 7/8	2 5/8	2.938	2.844	3.392	3.242	127/32	1.886	1.802	11/32	1.073	0.989
2	3 1/8	3.125	3.025	3.608	3.449	131/32	2.013	1.925	133/32	1.138	1.050
2 1/4	3 1/2	3.500	3.388	4.041	3.862	213/64	2.251	2.155	113/64	1.251	1.155
2 1/2	3 3/8	3.875	3.750	4.474	4.275	229/64	2.505	2.401	129/64	1.505	1.401
2 3/4	4 1/4	4.250	4.112	4.907	4.688	245/64	2.759	2.647	137/64	1.634	1.522
3	4 5/8	4.625	4.475	5.340	5.102	261/64	3.013	2.893	145/64	1.763	1.643
3 1/4	5	5.000	4.838	5.774	5.515	33/16	3.252	3.124	113/16	1.876	1.748
3 1/2	5 3/8	5.375	5.200	6.207	5.928	37/16	3.506	3.370	115/16	2.006	1.870
3 3/4	5 3/4	5.750	5.562	6.640	6.341	311/16	3.760	3.616	21/16	2.134	1.990
4	6 1/8	6.125	5.925	7.073	6.755	315/16	4.014	3.862	23/16	2.264	2.112

Figure 6-A4

Dimensions of Common Heavy Hex Nuts and Heavy Hex Jam Nuts  
Mn/DOT 3391.2A (ASTM A307)

APPENDIX 6-A (Continued)

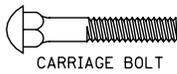
CARRIAGE BOLTS (LBS. PER HUNDRED) ①				
L (IN.)	DIAMETER			
	3/8"	1/2"	5/8"	3/4"
1	7.53	15.15		
1 1/4	8.13	16.24		
1 1/2	8.76	17.34	30.20	
1 3/4	9.52	18.44		
2	10.48	19.74	33.70	53.00
2 1/4	11.04	21.14		
2 1/2	11.80	22.44	37.80	58.30
2 3/4	12.56	23.84	39.90	
3	13.34	25.14	42.00	64.40
3 1/4	14.14	26.54		
3 1/2	14.84	27.84	46.20	70.40
3 3/4	15.64	29.14		
4	16.34	30.54	50.40	76.40
4 1/2	17.94	33.24	54.60	82.50
5	19.44	35.94	58.70	88.50
5 1/2	20.94	38.64	62.90	94.60
6	22.44	41.24	67.10	100.60
6 1/2	23.84	43.74	71.00	106.10
7	25.34	46.44	75.20	112.20
7 1/2	26.84	49.14	79.30	118.20
8	28.34	51.84	83.50	124.30
8 1/2	29.84	54.44		130.30
9	31.44	57.14	91.90	136.30
9 1/2	32.94	59.84		
10	34.44	62.54	100.30	149.30
11	37.44	67.94	108.70	
12	40.84	73.34	116.90	

STRUCTURAL BOLTS (LBS. PER HUNDRED) ①					
L (IN.)	DIAMETER				
	3/8"	1/2"	5/8"	3/4"	7/8"
1	7.31	15.76	27.06	43.76	
1 1/4	8.00	16.88	28.83	46.36	
1 1/2	8.78	18.13	30.60	48.96	73.62
1 3/4	9.56	19.51	32.56	51.56	77.19
2	10.33	20.89	34.72	54.42	80.76
2 1/4	11.11	22.27	36.87	57.53	84.66
2 1/2	11.88	23.65	39.03	60.64	88.89
2 3/4	12.66	25.23	41.18	63.75	93.12
3	13.44	26.41	43.34	66.86	97.35
3 1/4	14.21	27.79	45.49	69.97	101.58
3 1/2	14.99	29.17	47.65	73.08	105.81
3 3/4	15.76	30.55	49.80	76.19	110.04
4	16.54	31.93	51.96	79.30	114.27
4 1/2	18.09	34.69	56.27	85.52	122.73
5	19.64	37.45	60.58	91.74	131.19
5 1/2	21.20	40.21	64.89	97.96	139.65
6	22.75	42.97	69.20	104.18	148.11
6 1/2	23.74	44.24	72.20	108.40	154.70
7	25.34	46.94	76.40	114.40	162.70
7 1/2	26.84	49.54	80.60	120.30	170.70
8	28.34	52.24	84.80	126.30	178.70
8 1/2	29.84	54.94	89.00	132.30	186.70
9	31.34	57.64	93.10	138.30	195.70
9 1/2	32.94	60.34	97.30	144.30	
10	34.44	63.04	101.50	150.30	211.70
11	37.44	68.44	109.90	162.30	
12	40.44	73.74	117.90	175.30	228.70

ROUND WASHERS ②				
BOLT DIA.	HOLE DIA.	THICK. (IN.)	O.S. DIA.	LBS. PER 100 PCS.
1/4"	5/16"	0.065	3/4"	0.67
3/8"	7/16"	0.083	1"	1.49
1/2"	9/16"	0.109	1 3/8"	3.85
5/8"	1 1/16"	0.134	1 3/4"	7.70
3/4"	1 3/16"	0.148	2"	11.00
7/8"	1 5/16"	0.165	2 1/4"	15.40
1"	1 1/16"	0.165	2 1/2"	18.80
1 1/8"	1 1/4"	0.165	2 3/4"	22.10
1 1/4"	1 3/8"	0.165	3"	26.10
1 3/8"	1 1/2"	0.180	3 1/4"	
1 1/2"	1 5/8"	0.180	3 1/2"	38.46

① AMERICAN STANDARD PLAIN (MEDIUM)

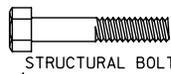
① CARRIAGE AND STRUCTURAL BOLTS INCLUDE NUTS. SEE SPEC. 3391.



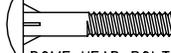
CARRIAGE BOLT



LAG SCREW



STRUCTURAL BOLT



DOME HEAD BOLT

NUTS (LBS. PER HUNDRED)		
BOLT DIA.	HEAVY HEX	HEAVY SQ.
1/4"	1.16	1.37
3/8"	3.14	3.74
1/2"	6.54	7.87
5/8"	11.9	14.3
3/4"	19.3	23.5
7/8"	29.7	36.2
1"	42.5	51.5
1 1/8"	59.2	72.4
1 1/4"	78.6	95.5
1 3/8"	102.0	125.0
1 1/2"	131.0	161.0

NAILS/SPIKES (LBS. PER HUNDRED)			
L (IN.)	SIZE	NAIL	SPIKE
1	2d	847	
1 1/4	3d	543	
1 1/2	4d	294	
1 3/4	5d	254	
2	6d	167	
2 1/4	7d	150	
2 1/2	8d	101	
2 3/4	9d	92	
3	10d	66	42
3 1/4	12d	61	39
3 1/2	16d	47	31
4	20d	29	22
4 1/2	30d	22	17
5	40d	17	13
5 1/2	50d	13	10
6	60d	10	9
7	5/16"		6
8	3/8"		4
9	3/8"		3
10	3/8"		3
11	3/8"		2

LAG SCREWS (LBS. PER HUNDRED)				
L (IN.)	DIAMETER			
	3/8"	1/2"	5/8"	3/4"
1	4.04			
1 1/4	4.67			
1 1/2	5.30	11.0		
1 3/4	5.93	12.1		
2	6.56	13.3	23.2	
2 1/2	7.82	15.5	26.8	41.8
3	9.10	17.8	30.4	47.1
3 1/2	10.4	20.0	34.0	52.3
4	11.6	22.3	37.6	57.5
4 1/2	12.9	24.6	41.2	62.7
5	14.1	26.9	44.7	68.0
5 1/2	15.4	29.1	48.3	73.2
6	16.6	31.4	51.9	78.4
6 1/2	17.9	33.6	55.5	83.6
7	19.1	35.9	59.0	88.9
7 1/2	20.4	38.2	62.6	94.1
8	21.6	40.5	66.2	99.3
9	24.1	45.0	73.4	110
10	26.6	49.5	80.6	120
11	29.1	54.0	87.8	131
12	31.6	58.5	95.0	141

DOME HEAD BOLTS (LBS. PER HUNDRED)					
L (IN.)	DIAMETER				
	1/2"	5/8"	3/4"	7/8"	1"
10	130	130	201		
11	138	138	213		
12	146	146	225	323	448
13	155	155	237	339	470
14	163	163	249	356	492
15	171	171	261	372	513
16	179	179	273	389	535
17	188	188	285	405	557
18	196	196	297	422	578
19	204	204	309	438	600
20	212	212	321	455	622
21	221	221	333	471	643
22	229	229	345	488	664
23	237	237	357	504	686
24	245	245	369	520	707
25	254	254	381	537	729
26	262	262	393	553	751
27	270	270	405	569	772
28	278	278	417	586	794
29	287	287	429	602	815
30	295	295	441	619	836



STUD DETAIL

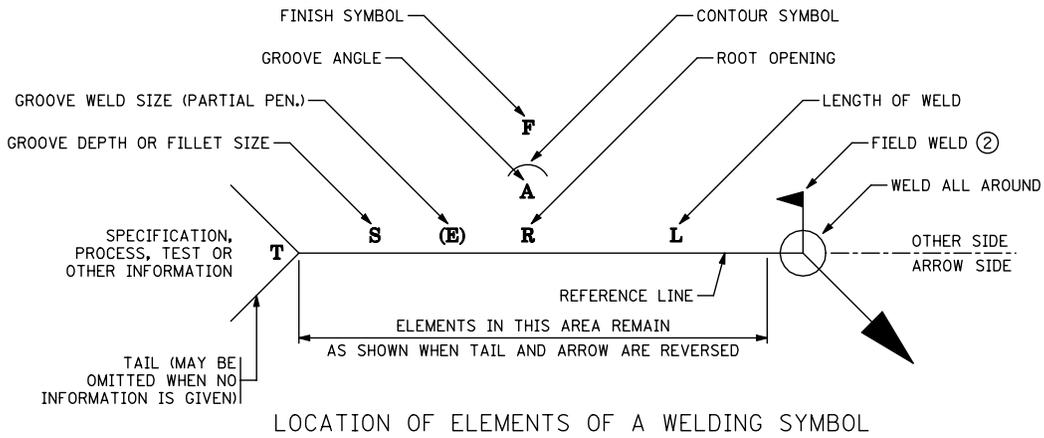
STUD SHEAR CONNECTORS (LBS. PER HUNDRED)					
STUD DIA.	3"	4"	5"	6"	7"
1/2"	21.0	27.0	33.0	39.0	45.0
5/8"	33.6	43.2	52.8	62.4	72.0
3/4"	49.0	61.5	74.0	86.5	99.0
7/8"	64.0	81.0	98.0	115.0	132.0

Figure 6-A5  
Hardware Details

APPENDIX 6-A (Continued)

BASIC ARC WELD SYMBOLS								
LOCATION SIGNIFIANCE	FILLET ①	PLUG OR SLOT	SQUARE	V	GROOVE BEVEL ①	FLARE V	FLARE BEVEL ①	BACK OR BACKING
ARROW SIDE								
OTHER SIDE								
BOTH SIDES								

SUPPLEMENTARY SYMBOLS AND INITIALS						
WELD ALL AROUND	FIELD WELD	CONTOUR		FINISH	TESTS	PROCESSES (PER SPEC. 2471)
		FLUSH	CONVEX	"G" = GRINDING "C" = CHIPPING "M" = MACHINING	"RT" = RADIOGRAPHIC "UT" = ULTRASONIC "MT" = MAGNETIC PARTICLE "PT" = DYE PENETRANT	"GMAW" = GAS METAL ARC "FCAW" = FLUX CORED ARC "SMAW" = SHIELDED METAL ARC "SAW" = SUBMERGED ARC



GENERAL NOTES

THE USE OF SYMBOLS MAY BE OMITTED WHEN SPECIFICATIONS AND PROCEDURES TO BE FOLLOWED ARE PRESCRIBED ELSEWHERE. GENERAL NOTES MAY BE PLACED ON A DRAWING SUCH AS: "UNLESS OTHERWISE INDICATED, ALL FILLET WELDS ARE 5/16" SIZE." WHEN BASIC WELD SYMBOLS ARE INADEQUATE, USE REFERENCES SUCH AS:



THE SIDE OF THE JOINT TO WHICH THE ARROW POINTS IS THE ARROW (OR NEAR) SIDE AND THE OPPOSITE SIDE OF THE JOINT IS THE OTHER (OR FAR) SIDE.

BASIC SYMBOLS AS PER AMERICAN WELDING SOCIETY.

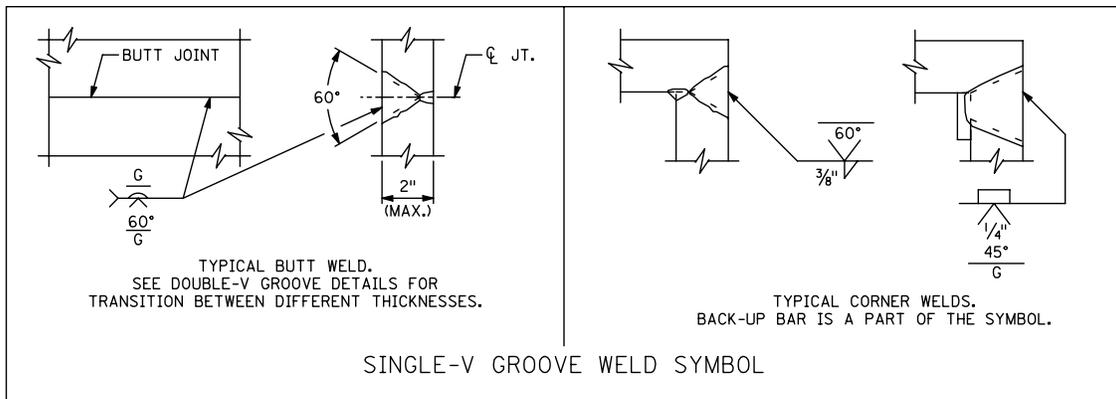
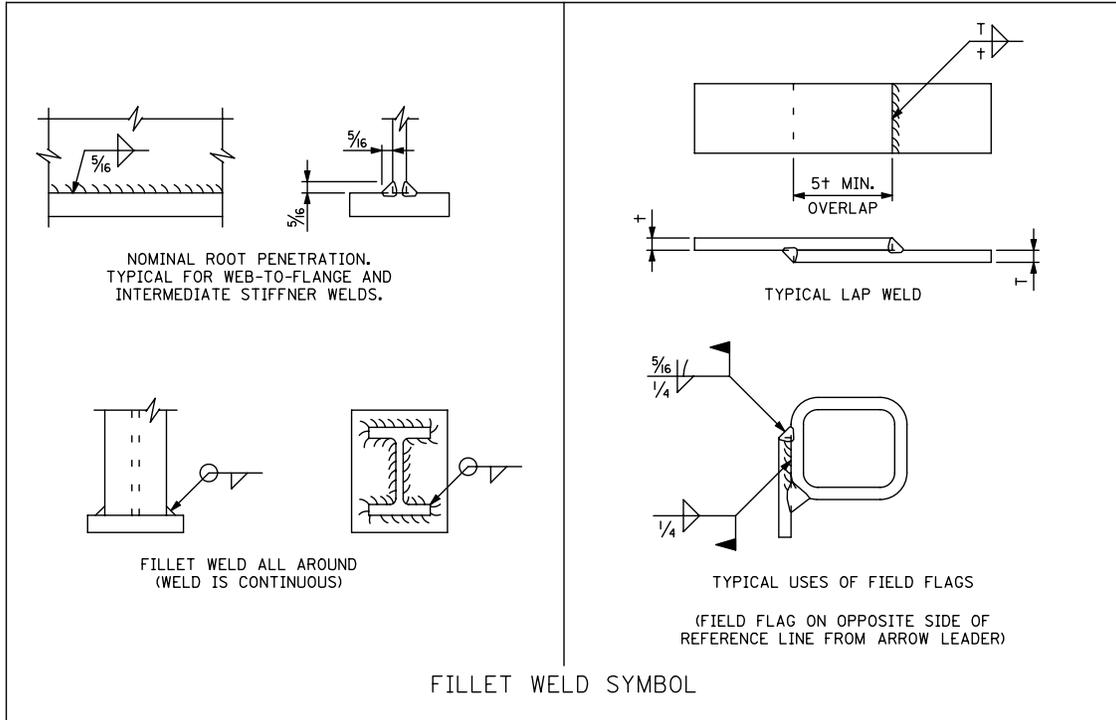
① VERTICAL LEG IS ALWAYS ON LEFT SIDE OF SYMBOL.

② FIELD WELD FLAG POINTS TOWARD TAIL OF WELD.

Figure 6-A6  
Welding symbols and Notes

APPENDIX 6-A (Continued)

APPLICATIONS OF WELDING SYMBOLS



GENERAL NOTES (Continued)

ARROW SIDE AND OTHER SIDE WELDS ARE THE SAME SIZE UNLESS OTHERWISE SHOWN.

SYMBOLS APPLY BETWEEN ABRUPT CHANGES IN DIRECTION OF JOINT OR AS DIMENSIONED (EXCEPT WHERE ALL-AROUND SYMBOL IS USED).

ALL WELDS ARE CONTINUOUS AND OF THE USERS STANDARD PROPORTIONS UNLESS OTHERWISE SHOWN.

FILLET WELDS MAY BE SHOWN GRAPHICALLY, THUS:

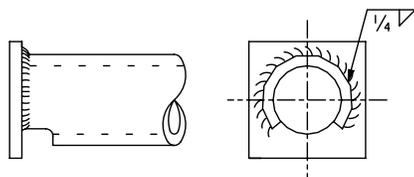


Figure 6-A7  
Welding Notes and Joints

APPENDIX 6-A (Continued)

APPLICATIONS OF WELDING SYMBOLS (Continued)

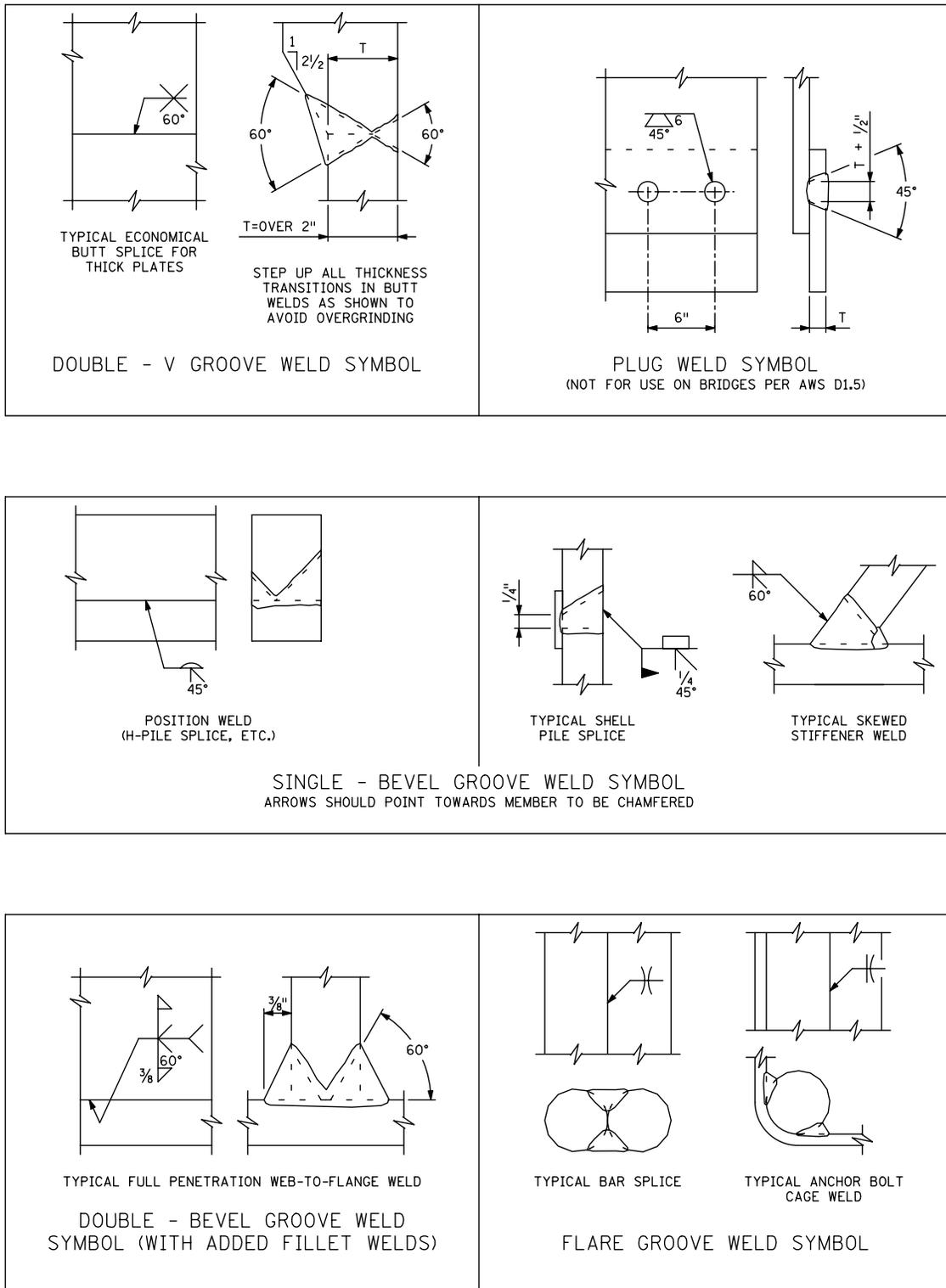


Figure 6-A8  
Welding Joints

## **8. WOOD STRUCTURES**

Wood is used for many bridge applications. It is used as a primary structural material for permanent bridges on secondary roads (e.g., decks, beams, and pile caps), and is used in temporary bridges on both secondary and major roads. It is often used for formwork and falsework on bridges with cast-in-place concrete elements. This section provides general design and detailing guidance for the LRFD design of longitudinal and transverse decks, glulam beams, and pile caps. It concludes with four design examples: a longitudinal spike laminated deck, a timber pile cap, a glulam beam superstructure, and a transverse deck on glulam beams. The transverse deck example goes through the design of two deck types, a transverse spike laminated and a transverse glulam. Wood bridge design is governed by the current edition of *AASHTO LRFD Bridge Design Specifications* including current interims, hereinafter referred to as AASHTO LRFD.

The design examples are followed by load rating examples for the elements designed in the design examples, except for the timber cap, because substructures are typically not load rated on new structures. Information on wood incorporated into the design of formwork and falsework can be found in the *MnDOT Bridge Construction Manual*. The construction of timber bridges is governed by *MnDOT Standard Specifications for Construction*, (MnDOT Std. Spec.,) Article 2403, Wood Bridge Construction.

### **8.1 Materials**

A variety of materials are incorporated into timber bridges, ranging from treated solid and laminated wood members to steel fasteners and hardware, as well as steel plates and shapes used as bracing or in connections.

This section briefly defines some commonly used terms for various wood materials:

#### *Lumber*

In general, lumber is defined as wood that is sawed, or sawed and planed.

In this chapter, lumber is commonly used in the term "dimension lumber", which is lumber that is nominal 2 to 4 inches thick on its edge, by 2 inches or more in width.

*Timber*

Timber is a term referring to larger pieces of lumber. For the purposes of this chapter the ASTM definition is applied, timber is lumber that is 5 inches thick and larger on its least dimension face.

*Wood*

The part of a tree inside of the bark, harvested and prepared for use as lumber and timbers to build structures; in the case of this section, constructing bridges. Specific species to be used are given in Article 8.1.1 below.

*Glulam Timber*

Glulam is short for "glued laminated" timber. Glued laminated timber is comprised of surfaced dimension lumber used as laminates and glued together in a factory to form larger timbers. The glulam timbers are commonly used for bridge beams and also for decks. The decks span either longitudinally between supports or transversely on beams. Frequency of glulam usage in decks varies by region around the country.

*Spike Laminated Decks*

Spike laminated decks are comprised of dimension lumber assembled in the shop to form deck panels, which are installed on supports in the field. Older spike laminated decks (generally 1970's and prior) were completely assembled in the field. Assembly (in the field or panels in the shop) consists of laying dimension lumber edgewise as laminates and driving large steel spikes through the wider faces of multiple layers of laminates in a pattern specified in AASHTO LRFD. These spike laminated decks are used transverse to the center line of road and supported on beams (deck thicknesses usually 6 to 8 inches thick measured vertically) or are used parallel to the centerline of road as longitudinal decks spanning between floor beams or substructures (deck thicknesses usually 8 to 18 inches thick). In AASHTO LRFD the term "spike laminated decks" is used, but these decks are sometimes also referred to as nail laminated or dowel laminated.

**8.1.1 Wood Products**

Structural wood products typically shall be visually graded West Coast Douglas Fir or Southern (Yellow) Pine as a standard. Other species should receive State Bridge Design Engineer approval prior to final design if it is intended to specify another species for use in a bridge. Refer to MnDOT Standard Spec., Art. 3426 Structural Wood. Designs should be based on the design values found in AASHTO LRFD. Design values not given in

AASHTO LRFD shall be obtained from the *National Design Specification for Wood Construction* (NDS).

**[Table 8.4.1.1.4-1]**

The AASHTO LRFD tabulated design values assume dry-use conditions. These tabulated values shall be modified if wood will be subject to wet use conditions. Table 8.1.1.1. has an abbreviated list of some typical design values for Douglas Fir-Larch, which is a common species used in bridges.

**Table 8.1.1.1 – Reference Design and Modulus of Elasticity Values**

Visually-Graded Sawn Lumber							
Species and Commercial Grade	Size Classification	Design Values (KSI)					
		F <sub>bo</sub>	F <sub>to</sub>	F <sub>vo</sub>	F <sub>cpo</sub>	F <sub>co</sub>	E <sub>o</sub>
Douglas Fir-Larch							
No. 1	Dimension* ≥ 2 in. Wide	1.00	0.675	0.18	0.625	1.50	1,700
Select Structural	B&S**	1.60	0.95	0.17	0.625	1.10	1,600
Select Structural	P&T***	1.50	1.00	0.17	0.625	1.15	1,600

**[8.2 - Definitions]**

\* Dimension Lumber Sizes, see AASHTO LRFD for definition

\*\* Beams and Stringers Sizes, see AASHTO LRFD for definition

\*\*\* Posts and Timbers Sizes, see AASHTO LRFD for definition

All wood members, that become part of the permanent bridge structure, should be treated with a preservative. Preservatives protect the wood against decay and organisms. Refer to Article 8.1.3 in this section for wood preservative information.

**[8.4.1.1.2]**

Lumber and timbers can be supplied in various finished sizes, depending on the sawing and planing done at the time of manufacture. Following are general definitions of some common finished sizes. Grading rules for specific species should be referenced if dimensions are important to the design for lumber that is not dressed (not planed), or surfacing can be specified as needed.

*Full sawn*

Sawed full to the specified size with no undersize tolerance allowed at the time that the lumber is manufactured.

*Rough sawn*

Lumber sawed to the specified size and not planed, and with small tolerances permitted under the specified size.

*Standard sawn*

Lumber sawed to size but not planed, and with minimum rough green sizes slightly less than rough sawn.

*Dressed lumber, or surfaced lumber (S4S, S1S, etc.)*

Lumber that has been sawed, and then surfaced by planing on one or more sides or edges. The most common is surfaced 4 sides (S4S). Sometimes if a specific dimension is needed by the design only 1 side is surfaced (S1S), or other combinations of sides and edges can be specified. Standard surfaced sizes can be referenced in the NDS.

The actual dimensions and moisture content used in the design should be indicated in the contract documents. MnDOT policy is to design for wet-use conditions (8.2.1 and 8.4.3).

**[Table 3.5.1-1]**

The design unit weight of most components is 0.050 kcf. Douglas Fir and Southern Pine are considered soft woods. For special designs using hard woods, the design unit weight is 0.060 kcf.

**[9.9.3.4]**

The coefficient of thermal expansion of wood parallel to its fibers is 0.000002 inch/inch/°F. AASHTO LRFD Article 9.9.3.4 provides design guidance on applicability of considering thermal effects.

**8.1.2 Fasteners and Hardware**

Structural steel elements incorporated into timber bridges must satisfy the strength and stability checks contained in Section 6 of the LRFD Specifications. For durability, generally all steel elements incorporated into timber bridges are hot-dipped galvanized. Compatibility of steel elements and hardware with the specified wood preservative shall be investigated. Some waterborne treatments actively corrode steel and hardware. Oil-type preservatives are generally compatible with steel and hardware and do not directly cause damage from reactivity. Use of uncoated steel (such as weathering steel) in wood bridges should be used with great caution to make certain durability is not compromised.

**8.1.3 Wood Preservatives**

Wood preservatives are broadly classified as oil-type or waterborne preservatives. All wood used in permanent structures must be treated with a preservative. Preservatives on the MnDOT approved list are to be specified for treated wood materials. Other preservative treatments can be used on an individual basis if a local agency conducts its own liability analysis for the preservative treatment proposed. Oil-type preservatives are not to be used where contact with pedestrians occurs. Preservatives used for pedestrian applications shall be safe for skin contact.

### **Oil-Type Preservatives**

The three most common oil-type preservatives that have been used in the past, or are currently being used in bridge applications are: creosote, pentachlorophenol, and copper naphthenate. The descriptions below are provided for general information only. As stated above, the MnDOT approved list shall be reviewed by the designer and owner. For bridge applications, oil-type preservatives are used almost exclusively for treating structural components. They provide good protection from decay, and provide a moisture barrier for wood that does not have splits. Because most oil-type treatments can cause skin irritations, they should not be used for applications that require repeated human or animal contact, such as handrails, safety rails, rub rails, or decks.

#### *Creosote*

Historically, creosote has been the most commonly used preservative in bridge applications in Minnesota. The high level of insolubles can result in excessive bleeding of the treatment from the timber surface, which can create a hazard when it contacts human skin. Creosote is an Environmental Protection Agency (EPA) restricted use pesticide. It should be noted that creosote is no longer on MnDOT's list of approved preservatives for the treatment of timber products.

#### *Pentachlorophenol*

As a wood preservative penta is effective when used in ground contact, in freshwater, or used above ground. Penta is difficult to paint and should not be used in applications subject to prolonged human or animal contact. Penta is an EPA restricted use pesticide. The penta producers have created guidance on the handling and site precautions with using this product.

#### *Copper Naphthenate*

Copper Naphthenate is effective when used in ground or water contact, and above ground. Unlike creosote and penta, Copper Naphthenate is not listed as a restricted use pesticide. However, precautions (dust masks, gloves, etc.) should be used when working with this wood treatment.

### **Waterborne Preservatives**

Waterborne preservatives are used most frequently for railings and floors on bridge sidewalks, pedestrian bridges and boardwalks, or other areas that may receive human contact. After drying, wood surfaces treated with these preservatives can also be painted or stained. Of the numerous waterborne preservatives, CCA, ACQ, and CA have been used in bridge

applications in the past. Each of these preservatives is strongly bound to the wood, thereby reducing the risk of chemical leaching.

*CCA (Chromated Copper Arsenate)*

CCA is an EPA restricted use pesticide that was generally used in the past to treat Southern Pine and other (easier to treat) wood species. The use of this product has been phased out because of environmental concerns with arsenic.

*EnviroSafe Plus®*

EnviroSafe Plus® is a borate based preservative treatment using Disodium Octaborate Tetrahydrate and a patented polymer binder. It contains no heavy metals, which can raise health, environmental, and disposal concerns. This treatment is not considered a problem for human contact, but it is not to be used for members in contact with the ground.

*ACQ (Alkaline Copper Quaternary)*

Multiple variations of ACQ have been standardized. ACQ was developed to meet market demands for alternatives to CCA. This product accelerates corrosion of metal fasteners. Hot dipped galvanized metal or stainless steel fasteners must be used to avoid premature fastener failure.

*MCA (Micronized Copper Azole)*

As the use of CCA was phased out, some wood suppliers began using CA waterborne preservatives, which evolved into the use of micronized CA (which uses micro sized copper particles). MCA treatments are considered to be less corrosive than CA and ACQ. However, at minimum to ensure durability, hot dipped galvanized hardware and steel should be used with MCA treated wood.

## **8.2 Timber Bridge Decks**

Wood or timber decks can be incorporated into a bridge in a number of different ways. Decks can be the primary structural element that spans from substructure unit to substructure unit or floor beam to floor beam, such as a longitudinal spike laminated deck.

Wood decks can also be secondary members used to carry vehicle or pedestrian loads to other primary members such as beams, stringers, or girders. As secondary members decks can be transverse spike laminated, transverse glulam, or simple transverse planks which are installed flatwise. Analysis modelling is described in 8.4.3.

**8.2.1 General  
Design and Detailing**

Section 9 of the AASHTO LRFD Specifications (Decks and Deck Systems) provides information on the design and detailing of decks. Designing specifically for wood decks is covered in Article 9.9. Some common longitudinal deck types are further described in Article 8.2.3 of this section.

**Applicability of Use**

AASHTO LRFD recommends limitations on the use of deck types as a guide to bridge owners and designers so that maintenance over the life of the bridge remains within expectations and does not become excessive.

**[C9.9.6.1]**

The use of spike laminated decks should be limited to secondary roads with low truck volumes, ADTT significantly less than 100 trucks per day.

**[C9.9.4.1]**

The recommended use for glulam decks is somewhat vague, but glulam decks should also be limited to secondary roads with low truck volumes. AASHTO LRFD states that this form of deck is appropriate only for roads having low to medium volumes of commercial vehicles.

**[9.9.2]**

Minimum thicknesses are specified in AASHTO LRFD for wood decks. The nominal thickness of wood decks other than plank decks shall not be less than 6.0 in. The nominal thickness of plank decks for roadways shall not be less than 4.0 in.

**[C9.9.7.1]**

Plank decks should be limited to low volume roads that carry little or no heavy vehicles. Plank decks do not readily accept and/or retain a bituminous surface. This deck type can sometimes be used economically on temporary bridges where wear course maintenance is less important. Thicker planks that provide higher capacity are economical if used or salvaged lumber can be incorporated into a temporary bridge.

In addition to reviewing applicability of a timber bridge based on traffic demands at the site, hydraulic considerations also need to be considered and the State Aid Bridge Hydraulic Guidelines must be followed in determining a low member elevation.

**Geometry**

Spike laminated timber deck panels should be laid out with panel widths that are multiples of 4 inches, which currently is the typical deck laminate width dimension. Glulam deck panels should be designed for standard laminate sizes based on the wood species. To facilitate shipping, deck panels should be detailed with plan widths less than 7'-6". Large and thick deck panels should have the lifting method and weight reviewed, to prevent damage to the wood.

**[8.4.4.3]****Moisture Conditions**

MnDot policy is for designs to be based on wet use conditions (>16% moisture content for glulam and >19% for sawn members). Applicable moisture factors are provided in AASHTO LRFD Table 8.4.4.3-1 for sawn lumber and 8.4.4.3-2 for glulam.

**[9.9.3.5]****Bituminous Wearing Surface**

AASHTO LRFD Article 9.9.3.5 requires a wearing surface conforming to Article 9.9.8 on wood decks. AASHTO LRFD Article C9.9.8.1 recommends bituminous wearing surfaces for timber decks, except for decks consisting of planks installed flatwise that will not readily accept and/or retain a bituminous wearing surface. It also recommends that deck material be treated using the empty cell process followed by an expansion bath or steaming. The bituminous wearing course should have a minimum compacted depth of 2 inches. For proper drainage, MnDOT recommends a cross slope of 0.02 ft/ft whenever practicable. The Spike Laminated Decks section below includes some discussion pertaining to maintenance of bituminous wearing surface, which has some applicability to all deck types.

**[C9.9.7.1]****[9.9.8.2]****8.2.2 Loads****Dead Load**

MnDOT uses a unit weight of 0.150 kcf for the bituminous wearing surface dead load (MnDOT Table 3.3.1). A 0.020 ksf dead load is to be included in all designs in order to accommodate a possible future wearing surface. The timber rail system is equally distributed across the deck, or equally to all beams.

**[3.6.1/3.6.2.3]****Live Load**

Live load and live load application shall be in accordance with AASHTO LRFD. Dynamic load allowance need not be applied to wood components.

**[9.9.3.1]**

For timber structures with longitudinal flooring, the live load shall be distributed using the appropriate method. Glulam and spike laminated are discussed below including under the spreader beam section because the appropriate method will typically require the use of a spreader beam. Transverse and longitudinal decks with planks installed flatwise (wood plank decks) are discussed in AASHTO LRFD Article 4.6.2.1.3. Tire contact area and dimensions are defined in LRFD Article 3.6.1.2.5.

**8.2.3 Longitudinal  
Wood Decks**

Three types of wood decks that function as primary structural elements spanning longitudinally are used in Minnesota; glulam panels, stress laminated decks, and spike laminated decks. However, stress-laminated

decks are considered non-standard and the design approach should receive approval from the State Bridge Design Engineer prior to final design. Calculations with validation are required for non-standard designs. Approval should also be obtained for other less common deck types and for less common materials, such as Parallel Strand Lumber (PSL), Fiber Reinforced Polymer wood (FRP), or wood species other than Douglas Fir or Southern (Yellow) Pine.

In addition, skews over 20° require special consideration and coordination with the State Bridge Design Engineer to assure proper support for the top of the abutments to prevent superstructure instability, and to confirm the method of analysis for the longitudinal deck. Individual designs may require more or less attention depending on magnitude of skew, abutment type (concrete or timber), abutment height, soil conditions, etc.

To prevent movement of the deck panels in the completed structure, positive attachment is required between the panels and the supporting component (See Article 8.2.5 of this manual).

**[9.9.4]****Glulam Decks**

Glulam wood deck panels consist of a series of panels, prefabricated with water-resistant adhesives, which are tightly abutted along their edges. Stiffener beams, or spreader beams, are used to ensure load distribution between panels. It is recommended to obtain approval on the design approach for this deck type since it is not a common design in Minnesota.

**[9.9.5]****Stress Laminated Decks**

Stress laminated decks consist of a series of wood laminations that are placed edgewise and post-tensioned together, normal to the direction of the lamination.

**[9.9.5.6]**

In stress laminated decks, with skew angles less than 25°, stressing bars should be detailed parallel to the skew. For skew angles between 25° and 45°, the bars should be detailed perpendicular to the laminations, and in the end zones, the transverse prestressing bars should be fanned in plan or arranged in a step pattern. Stress laminated decks should not be used for skew angles exceeding 45°. AASHTO LRFD Article 9.9.5 contains design and detailing guidance for stress laminated decks.

**[9.9.6]****Spike Laminated Decks**

Spike laminated decks consist of a series of dimension lumber laminations that are placed edgewise between supports and spiked together on their wide face. The laminated deck is prefabricated at a

plant in panels that are shipped to the site. The connection between adjacent panels most commonly used in current industry practice is a ship-lap joint, but AASHTO LRFD does not directly give credit to the ship-lap joint for transfer of wheel loads. In accordance with AASHTO LRFD, spreader beams are required to ensure proper load distribution between panels (see below). The laminates are treated with preservative after drilling pilot holes for the spikes, and prior to assembling and installing spikes in the panels. Butt splicing of laminations within their unsupported length is not allowed.

The use of these decks is limited to secondary roads with low truck volumes (i.e. ADTT significantly less than 100 trucks per day). Frequent heavy truck loading may increase bituminous cracking resulting in accelerated bituminous deterioration and increased maintenance. To reduce future bituminous maintenance, the owner could elect to over design the deck or incorporate the use of geotextiles in the bituminous wearing surface. Waterproofing may be considered, but careful attention to details is required to avoid direct contact between fresh oil-type treatments and rubberized water proofing, to prevent degradation of the waterproofing membrane which results in liquidation of the membrane.

#### **[4.6.2.3]**

#### **Spreader Beams**

Spreader beams, or transverse stiffener beams, are attached to the underside of longitudinal glulam and spike laminated decks as a method for panels to be considered interconnected by design.

AASHTO LRFD Table 4.6.2.3-1 shows a schematic for longitudinal laminated decks (glulam and spike laminated). AASHTO LRFD requires spans exceeding 15.0 feet to be designed according to the provisions of Article 4.6.2.3, which includes the use of spreader beams. AASHTO LRFD Article 9.9.4.3 gives minimum spreader (or stiffener) beam requirements. The rigidity,  $EI$ , of each spreader beam cannot be less than 80,000 kip-in<sup>2</sup>. The spreader beams must be attached to each deck panel near the panel edges and at intervals not exceeding 15.0 inches. The spreader beam spacing is not to exceed 8.0 ft.

Research has shown spreader beams to be effective in transferring load between panels and the spreader beams stiffen longitudinal decks in the transverse direction. One such research project by the University of Minnesota that was published in January 2003 used 6 inch wide x 12 inch deep spreader beams which are a common industry standard. MnDOT approves of using 6 inch wide x 12 inch deep spreader beams at the AASHTO specified maximum spreader beam spacing of 8 feet. Closer

spacing can be used to reduce bituminous cracking, including on an existing bridge.

**[9.9.3]**

Decks with spans 15.0 feet and less may be designed by one of the three methods given in AASHTO LRFD. The simplest method is Article 4.6.2.1. However, experience has shown that this method may result in thicker decks compared to other methods. If approved by the State Bridge Design Engineer on a per project basis, spans 15.0 feet and less could be designed by Article 4.6.2.3, which includes the use of a spreader beam.

**8.2.4 Design/  
Analysis**

Most longitudinal wood decks will be designed per AASHTO LRFD Article 4.6.2.3 and incorporate the use of spreader beams. Exterior strips or edge beams are not specifically designed for on timber deck bridges with spreader beams. MnDOT designs are performed on a unit strip one foot wide. Manipulate the code values (invert and multiply by 12) to determine distribution factors on a per foot basis.

MnDOT design span lengths are center to center of bearing at support for the longitudinal wood member being designed. This simplification was adopted in response to what designers in the local industry generally use.

The maximum span length for a given deck thickness is dependent on several factors including: superstructure type, wood species and grade, deck width, and live load deflection. Table 8.2.4.1 provides typical deck thicknesses and design span lengths for various longitudinal deck configurations. Table 8.2.4.2 contains typical design span lengths for longitudinal spike laminated deck thicknesses ranging from 10 to 18 inches. Actual design span lengths must be verified with calculations for the species and grade of wood used in a particular deck.

**Table 8.2.4.1 – Typical Designs Spans for Various Longitudinal Timber Deck Systems**

Superstructure Type	Deck Thickness (in)	Design Span Length (ft)
Sawn Lumber Deck Systems		
Spike-Laminated	10-18	10-35
Stress-Laminated	10-18	10-35
Glulam Deck Systems		
Standard Panel	8-16	10-37
Post-Tensioned	9-24	10-50

**Table 8.2.4.2 – Typical Span Lengths for Longitudinal Spike Laminated Sawn Deck Thicknesses**

Deck Thickness (in)	Typical Max. Design Span Length (ft)
10	≤ 10
12	≤ 17
14	≤ 25
16	≤ 31
18	≤ 35

### **Load Distribution and Modeling**

All spans are designed as simple spans. Check bending of deck using size factor, if applicable. Also check deflection, horizontal shear, and compression perpendicular to the grain.

### **8.2.5 Detailing**

#### **[9.9.4.2]**

#### **[9.9.5.5]**

Typically metal plate connectors are used to attach longitudinal deck panels to pile caps at piers to engage the deck in each span. Lag screws or deformed shank spikes can be used through the metal plate connectors down to wood supports. At minimum, detail no less than two metal tie-down plates per deck panel. The spacing of the tie-downs along each support shall not exceed 3.0 feet for stress laminated decks. Tie-downs at abutments shall have the same quantity and spacing requirements, but metal plates are not required unless large washers are determined as needed by the designer.

AASHTO LRFD provides guidance for longitudinal deck tie-downs based on standard practice for glulam and spike laminated decks, and higher strength tie-down for stress laminated decks. The designer shall consider individual site conditions (such as design flood elevation and possible buoyancy forces) to make the determination as to if tie-downs are adequate for a specific structure. The USDA Forest Service recommends through bolting from the superstructure to substructure with timber cap beams, and grouted anchors if concrete substructures are used.

#### **[9.9.6.1]**

The requirements in Article 9.9.6.1 of AASHTO LRFD are to be followed for spike placement in spike laminated decks. Spikes shall be of sufficient length to totally penetrate four laminations, and placed in lead holes through pairs of laminations at intervals not greater than 12.0 inches in an alternating pattern top and bottom. (AASHTO Figure 9.9.6.1-1). Laminations shall not be butt spliced within their unsupported length. Drive spike spacing at ship-lap joints is calculated by the designer.

### **8.3 Timber Bridge Superstructures**

Wood components can be and have been incorporated into bridge superstructures in a wide variety of applications. Article 8.2 outlined several different deck types that can span longitudinally from substructure to substructure or from floor beam to floor beam. The longitudinal spike laminated deck was the most common timber bridge type constructed in Minnesota for many years, and a large number of these bridges remain in existence.

The most common timber bridge type in Minnesota for longer spans consists of glulam beams with transverse wood decks. In Minnesota, the transverse decks on glulam beams traditionally have been spike laminated. Transverse glulam decks recently have become more common for some newer installations. Nationwide, transverse glulam decks are the more common deck type on glulam beams. The analysis and detailing of this bridge type is not complex and a design example is provided in this section. Transverse wood decks are also used on sawn beams, but in the span ranges that sawn timber beams can be used longitudinally, spike laminated deck superstructures currently are usually more economical. Many sawn beam bridges remain in existence around Minnesota.

Wood is also used in hybrid superstructures. The most common is transverse wood decking on steel beams. Although this superstructure type is currently considered non-standard for new permanent bridge installations with State funding, it is commonly used for temporary bridges. It is also used for bridges on very low volume roads and private bridges.

Other less common hybrids and configurations exist for timber bridge superstructures. Special designs incorporating wood components are sometimes desired for aesthetic purposes, especially in span lengths that traditionally accommodate wood members. Once again, if considering non-standard superstructure types, the design approach should receive approval from the State Bridge Design Engineer prior to final design. Some examples of special designs that increase strength of timber components are transverse post-tensioned glulam beams with a laminated deck and fiber reinforced polymer glulam beams (FRP). Examples of special designs with increased aesthetic appeal are glulam girder or arch spans, and wood truss spans.

#### **8.3.1 Camber / Deflections**

MnDOT does not require wood decks to be fabricated with specific camber values. During fabrication of panels, if there is any natural camber of the deck it should be planned to be placed up to reduce the

appearance of sag in a span. Longitudinal panels comprised of glulam laminates spiked together can reach longer span lengths and may need to be designed with camber. Design glulam beams for camber of dead load deflection plus long term creep.

#### **8.4 Timber Pile Caps/Substructures**

Timber pile caps are most commonly used for timber bridges, supported on cast-in-place piles. As a standard, large sawn timbers are used for caps. Special designs sometimes use glulam caps. Due to the low stiffness of timber caps that are relatively slender, equal load distribution to the piles supporting the cap is not to be assumed when calculating pile loads. A continuous beam model similar to that used for analyzing the cap to determine reactions (see Art. 8.4.3 below), is to be used when calculating the loads for the piles supporting a timber cap.

##### **8.4.1 Substructure Details**

Typically, 12 inch cast-in-place piles are to be used in abutments, and 16 inch cast-in-place piles are to be used in piers unless project specific approval is obtained. MnDOT does not allow the use of timber piles for main structural support (support of caps). Timber piles may be used at wingwall ends. If soil conditions do not allow the use of cast-in-place piles, steel H-piles with special details may be used. If H-piles are used, all pier piles shall be encased in pile shells.

To prevent uplift and movements, pile caps must have positive attachment to the piles. Similar to detailing for decks, the designer shall review individual site conditions and determine adequate cap to pile connections. Consider using concrete caps at sites with high debris, ice jams, or potentially high buoyancy forces. Concrete caps can be painted brown if desired for aesthetic reasons. In reviewing site conditions, the State Aid Bridge Hydraulic Guidelines must be followed, and pile embedment and unsupported length considering scour also need to be evaluated.

##### **8.4.2 Geometry**

MnDOT's standard timber abutment is 4 foot maximum clear height on the front face from ground elevation to bottom of superstructure. Tie backs for abutments are not standard. Backing planks are normally 3 inch x 12 inch or 4 inch x 12 inch. The designer shall verify backing plank size and pile spacing based on at-rest earth pressure. Passive pressure used for concrete abutment design need not be considered since timber abutments are less rigid, and wood bridges have negligible temperature expansion. Other abutment configurations, dimensions, or with tie-backs (which may be required, for example, on larger skews) are

to receive approval by the State Bridge Design Engineer prior to final design.

The standard timber size for abutment pile caps is 14 inch x 14 inch. Pier pile caps are 16 inch x 16 inch. Designers should use a maximum length of 36 feet for cap timbers, or verify availability of longer lengths. This constraint may require a splice in the pile cap. If a splice is necessary, it should be located over an internal pile.

#### **8.4.3 Design / Analysis**

Design for a wet-use condition.

For design of the cap, assume that the railing weight is uniformly distributed across the cap.

When analyzing pile caps and transverse decks use three different models:

- 1) a simply supported span in determining the positive bending moment
- 2) a fixed-fixed span in determining the negative bending moment
- 3) a continuous beam (with a hinge to represent a splice) in determining the shear forces and reactions

The third model requires the live load to be placed at various locations along the span to determine the critical member forces. This is illustrated in the design examples.

#### **8.4.4 Camber / Deflections**

Timber pile caps are not cambered. Deflection normally does not control the design of a cap due to the short design spans.

#### **8.5 Railings**

Railings used on timber bridges shall be crash tested rail systems for the appropriate application; such as longitudinal timber deck, transverse timber deck on beams, etc. Timber railings are sometimes used on concrete decks for aesthetic reasons, and standard plans of crash tested railings for this application are also available.

In general, rail systems must conform to the requirements of Section 13 of the AASHTO LRFD and crash tested in accordance with *NCHRP Report 350 Recommended Procedures for the Safety Performance Evaluation of Highway Features*.

Crash tested timber railing systems can be found on the FHWA website:  
[http://safety.fhwa.dot.gov/roadway\\_dept/policy\\_guide/road\\_hardware/barriers/bridgerailings/docs/appendixb7h.pdf](http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/docs/appendixb7h.pdf)

[http://safety.fhwa.dot.gov/roadway\\_dept/policy\\_guide/road\\_hardware/barriers/bridgerailings/docs/appendixb5.pdf](http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/docs/appendixb5.pdf)

Standard plan sheets are available on the USDA Forest Services Website: [www.fpl.fs.fed.us](http://www.fpl.fs.fed.us) A search for "standard plans" produces many standard plans related to timber bridges, including for crash tested rail systems that were created under a cooperative effort including the University of Nebraska–Lincoln, the USDA Forest Service, Forest Products Laboratory, and FHWA.

### **[13.7.2]**

In addition to a crash tested rail system for the proper bridge superstructure configuration, the rail system must be crash tested at the proper Test Level for the bridge traffic usage. Test Level selection criteria can be found in Article 13.7.2 of AASHTO LRFD, and Table 13.7.2-1 has crash test criteria.

Section 13 of this Manual covers bridge railings and barriers. Article 13.2.1 gives requirements based on speed.

### **8.6 Additional References**

Additional wood design information for use in designing wood bridges is available in the following references:

- 1) *National Design Specifications – Wood Construction* (NDS)
- 2) *Timber Construction Manual* (AITC)
- 3) Ritter, M.A., *Timber Bridges, Design, Construction, Inspection and Maintenance*, EM7700-B. Forest Service, U.S. Department of Agriculture, Washington, D.C., 1990
- 4) *National Conference on Wood Transportation Structures* (NCWTS)
- 5) AASHTO LRFD 8.14 has an extensive list of References

### **8.7 Design Examples**

Article 8.7 demonstrates the design of multiple bridge elements in accordance with AASHTO LRFD through several design examples. The design examples include a longitudinal spike laminated deck, a timber pile cap on pier piling, a glulam beam superstructure, and the transverse deck on the glulam beams. The transverse deck example goes through the design of two different deck types, a transverse spike laminated and a transverse glulam.

**8.7.1 Longitudinal  
Spike Laminated  
Timber Deck  
Design Example**

This first example goes through the design of a longitudinal spike laminated timber bridge deck. There are no longitudinal girders in the bridge, and so this bridge type is also sometimes generically referred to as a timber slab span. It should be noted that these bridge decks are usually reserved for secondary roads with low truck traffic volumes.

The deck panel span under investigation is an "interior" strip of an intermediate span, which spans from one pile cap to another pile cap. Refer to Figure 8.7.1.1 which shows the general layout and dimensions. In addition, Article 8.7.2 of this manual contains the example design of the timber pile cap which provides support bearing for the beginning and end of this longitudinal deck span.

**A. Material and Design Parameters**

**[Figure 8.3-1]**

The dimension annotations used throughout this design example are as follows. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications*, except for sawn lumber descriptive names. The letter notations will be used in this example (b, d, etc.).

Nominal dimensions of sawn lumber are always used for dead load calculations. The dimensions used for calculating member capacity need to be determined for each individual case depending on the actual surfacing specified and supplied. These are commented on below.

**[8.4.1.1]**

**1. Pile Cap**

Width of the pile cap member =  $b_{cap} = 16$  in

Depth of the pile cap member =  $d_{cap} = 16$  in

16 inch x 16 inch pile caps are supplied as rough sawn. For rough sawn, MnDOT allows the use of these dimensions as actual (for rough sawn, slight tolerances are permitted at the time of manufacture). The validity of the pile cap dimensions used here will be later checked in Article 8.7.2 of this manual.

**[9.9.8]**

**2. Bituminous Wearing Surface**

MnDOT uses a 2% cross slope whenever practicable. In this case, a minimum thickness of 2 in at edge of roadway (face of curb) and 6 in thickness at centerline of the road gives an average depth of wearing course = 4 in. Therefore, the bituminous wearing course thickness used for dead load calculations =  $d_{ws} = 4$  in.

**3. Curb and Railing** [TL-4 Glulam Timber Rail with Curb]

Width of timber curb =  $b_{\text{curb}} = 12$  in

Depth of timber curb =  $d_{\text{curb}} = 6$  in

Width of timber rail post =  $b_{\text{post}} = 10$  in

Length of timber rail post =  $L_{\text{post}} = 8$  in

Depth of timber rail post =  $d_{\text{post}} = 47$  in

Width of timber spacer block =  $b_{\text{spacer}} = 4.75$  in

Length of timber spacer block =  $L_{\text{spacer}} = 8$  in

Depth of timber spacer block =  $d_{\text{spacer}} = 13.5$  in

Width of timber scupper =  $b_{\text{scupper}} = 12$  in

Length of timber scupper =  $L_{\text{scupper}} = 48$  in

Depth of timber scupper =  $d_{\text{scupper}} = 8$  in

Width of timber rail =  $b_{\text{rail}} = 6.75$  in

Depth of timber rail =  $d_{\text{rail}} = 13.5$  in

Spacing between barrier posts =  $s_{\text{post}} = 6.25$  ft = 75 in (maximum)

The timber barrier design is not a part of this design example, but the dimensions are used for weight considerations. Refer to the resources noted in Article 8.5 of this manual for TL-4 crash tested bridge rail details.

**[8.4.1.1, 9.9.2]****4. Deck Laminates**

Assumed depth of timber deck panel laminates =  $d_{\text{lam}} = 14$  in

Assumed width of timber deck panel laminates =  $b_{\text{lam}} = 4$  in

**[8.4.1.1.2]**

Visually-graded longitudinal deck panel lumber is normally supplied rough sawn and surfaced on one side so that panels can be fabricated to the specified dimensions. The nominal dimensions are used for both dead load calculations and section properties for member capacity because the effective net dimensions can be considered the same as the nominal dimensions in the overall finished deck panels. This is true for a longitudinal spike laminated deck with the many individual laminates, if they are made up of rough sawn lumber.

**5. Span Lengths**

Actual longitudinal length of deck panels, which for an intermediate bridge span is also the distance between the centerlines of adjacent supporting pile caps, are usually in multiples of two feet which is how the lumber is supplied.

$$L = 22.0 \text{ ft}$$

MnDOT uses the effective span, or design span, as center to center of the deck bearing length on each cap.

Because of the end/end deck placement on the pier caps, the intermediate span of the longitudinal deck panels in a multi-span bridge has the longest effective span,  $L_e$ .

$$L_e = L - \frac{1}{2} \cdot b_{\text{cap}} = 22.0 - \frac{1}{2} \cdot \frac{16}{12} = 21.33 \text{ ft}$$

Figure 8.7.1.1 illustrates the basic layout and dimension used in the design.

## 6. Unit Weights and Moisture Content

Type of deck panel wood material = Douglas Fir-Larch (No.1)

**[Table 3.5.1-1]**

**[MnDOT Table 3.3.1]**

**[MnDOT 3.3]**

Unit weight of soft-wood =  $\gamma_{\text{DFL}} = 0.050 \text{ kcf}$

Unit weight of bituminous wearing surface =  $\gamma_{\text{ws}} = 0.150 \text{ kcf}$

Standard MnDOT practice is to apply a future wearing course of 20 psf.

**[8.4.1.1.3]**

Moisture content (MC) of timber at the time of installation shall not exceed 19.0%

MnDOT designs for in-service wet-use only which is a MC of greater than 19% for sawn lumber.

## 7. Douglas Fir-Larch Deck (No. 1) Strength Properties

**[Table 8.4.1.1.4-1]**

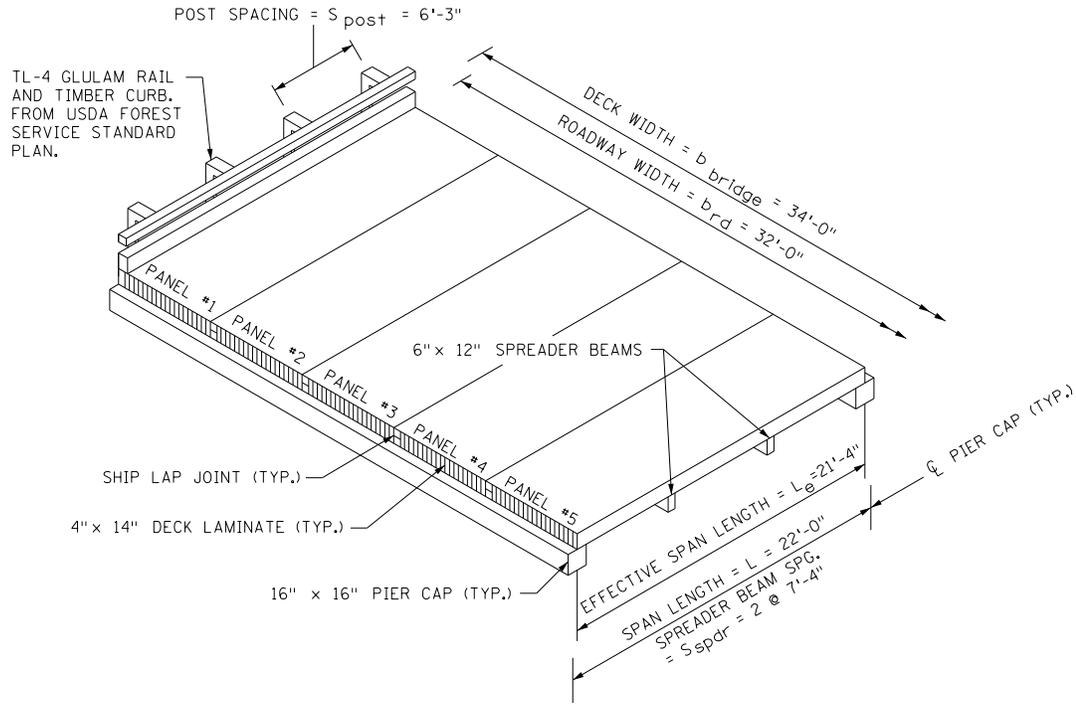
Reference Design Value for flexure =  $F_{b0} = 1.00 \text{ ksi}$

Reference Design Value for compression perpendicular to the grain

=  $F_{c_{p0}} = 0.625 \text{ ksi}$

Modulus of elasticity =  $E_o = 1700 \text{ ksi}$

Note:  $F_{c_{p0}}$  shown for the deck lumber is equal to or less than for the cap, so for the Bearing Strength check,  $F_{c_{p0}} = 0.625 \text{ ksi}$  for the deck lumber will be used.



**Figure 8.7.1.1 – Longitudinal Timber Deck Layout\***

\*For clarity, the timber curb/railing on the near side and the bituminous wearing surface are not shown.

**Select the Basic Configuration**

The bridge deck consists of 5 deck panels that are designed as interconnected, and are oriented parallel to traffic. The laminates of each panel are connected using horizontal spikes. The panels are attached to each other using vertical spikes through ship lap joints, and transverse stiffener beams, also called spreader beams, provide the interconnection per AASHTO LRFD.

The deck panel depth and spreader beam sizes are based on deflection limits as well as strength considerations. The interconnection provided by the spreader beams enable the longitudinal deck panels to act as a single unit under deflection. In addition, each spike laminated deck span is designed as a simply supported member.

**A. Deck Panel Widths**

The deck panel sizes are given here to clarify the sketches contained throughout this design example.

- Width of bridge deck panel #1 =  $b_1 = 7.33$  ft
- Width of bridge deck panel #2 =  $b_2 = 6.33$  ft

Width of bridge deck panel #3 =  $b_3 = 6.67$  ft  
 Width of bridge deck panel #4 =  $b_4 = 6.33$  ft  
 Width of bridge deck panel #5 =  $b_5 = 7.33$  ft  
 Overall width of bridge deck =  $b_{\text{bridge}} = \sum(b_{\#}) = 34.0$  ft

Width of each timber barrier =  $b_{\text{barrier}} = 1.0$  ft

Width of roadway =  $b_{\text{rd}} = b_{\text{bridge}} - 2 \cdot b_{\text{barrier}} = 34.0 - (2 \cdot 1) = 32.0$  ft

### [9.9.6.3]

#### B. Spreader Beam Dimensions

For interconnection of the deck panels, the spreader beam dimensions that MnDOT uses, based on research (refer to Art. 8.2.3), are as follows:

Width of spreader beams =  $b_{\text{spdr}} = 6$  in  
 Depth of spreader beams =  $d_{\text{spdr}} = 12$  in

### [9.9.4.3.1]

The size of the spreader beam exceeds the minimum specified in AASHTO LRFD. The spreader beams will be further investigated later in this example.

### Determine Dead and Live Load Bending Moments

#### A. Dead Loads per Unit Strip (1 ft)

The units for the dead load results are given in kips for a 1 ft wide longitudinal strip.

**1. Dead Loads per Longitudinal Foot** (these units could also be given as kips per square foot).

Weight of deck =  $w_{\text{deck}} = \gamma_{\text{DFL}} \cdot d_{\text{lam}} = 0.050 \cdot 14/12 = 0.058$  klf/ft

Weight of wearing surface =  $w_{\text{ws}} = \gamma_{\text{ws}} \cdot d_{\text{ws}} = 0.150 \cdot 4/12 = 0.050$  klf/ft

Weight of future wearing course =  $w_{\text{FWC}} = 0.020$  klf/ft

#### 2. Determine Linear Weight of Rail System Elements

Volume of timber curb per foot of bridge length =  $v_{\text{curb}}$

$$v_{\text{curb}} = (b_{\text{curb}} \cdot d_{\text{curb}} \cdot 12 \text{ in/ft}) = (12 \cdot 6 \cdot 12) = 864.0 \text{ in}^3/\text{ft}$$

Volume of rail post and spacer block per foot of bridge length =  $v_{\text{post}}$

$$v_{\text{post}} = [b_{\text{post}} \cdot L_{\text{post}} \cdot d_{\text{post}} + b_{\text{spacer}} \cdot L_{\text{spacer}} \cdot d_{\text{spacer}}] / s_{\text{post}}$$

$$v_{\text{post}} = [(10 \cdot 8 \cdot 47) + (4.75 \cdot 8 \cdot 13.5)] / 6.25 = 683.7 \text{ in}^3/\text{ft}$$

Volume of scupper per foot of bridge length =  $v_{\text{scupper}}$

$$v_{\text{scupper}} = (b_{\text{scupper}} \cdot L_{\text{scupper}} \cdot d_{\text{scupper}}) / s_{\text{post}}$$

$$v_{\text{scupper}} = (12 \cdot 48 \cdot 8) / 6.25 = 737.3 \text{ in}^3/\text{ft}$$

Volume of timber rail per foot of bridge length =  $v_{\text{rail}}$

$$v_{\text{rail}} = (b_{\text{rail}} \cdot d_{\text{rail}} \cdot 12 \text{ in/ft}) = (6.75 \cdot 13.5 \cdot 12) = 1093.5 \text{ in}^3/\text{ft}$$

Volume of timber railing per longitudinal foot of bridge length =  $v_{\text{barrier}}$

$$v_{\text{barrier}} = v_{\text{curb}} + v_{\text{post}} + v_{\text{scupper}} + v_{\text{rail}}$$

$$v_{\text{barrier}} = 864 + 683.7 + 737.3 + 1093.5 = 3378.5 \text{ in}^3/\text{ft}$$

$$v_{\text{barrier}} = 3378.5/12^3 = 1.955 \text{ ft}^3/\text{ft}$$

Total linear weight of combined timber curbs/railings =  $w_{\text{barrier}}$

$$w_{\text{barrier}} = \frac{2 \cdot \gamma_{\text{DFL}} \cdot v_{\text{barrier}}}{b_{\text{bridge}}} = \frac{2 \cdot 0.050 \cdot 1.955}{34.0} = 0.006 \text{ klf}$$

This linear weight result assumes that the curb/railing weight acts uniformly over the entire deck width.

### 3. Spreader Beam Point Loads on 1 ft Wide Longitudinal Strip

$$\text{Area of spreader beam} = A_{\text{spdr}} = d_{\text{spdr}} \cdot b_{\text{spdr}} = (12 \cdot 6)/144 = 0.5 \text{ ft}^2$$

$$\text{Spreader beam load} = P_{\text{spdr}} = \gamma_{\text{DFL}} \cdot A_{\text{spdr}} = 0.050 \cdot 0.50 = 0.025 \text{ kips/ft}$$

[AISC 14<sup>th</sup> p. 3-213]

### B. Dead Load Bending Moments per Unit Strip (1 ft)

Maximum bending moment due to deck weight =  $M_{\text{deck}}$

$$M_{\text{deck}} = \frac{w_{\text{deck}} \cdot (L_e)^2}{8} = \frac{0.058 \cdot 21.33^2}{8} = 3.30 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to wearing surface weight =  $M_{\text{ws}}$

$$M_{\text{ws}} = \frac{w_{\text{ws}} \cdot (L_e)^2}{8} = \frac{0.050 \cdot 21.33^2}{8} = 2.84 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to future wearing surface weight =  $M_{\text{FWC}}$

$$M_{\text{FWC}} = \frac{w_{\text{FWC}} \cdot (L_e)^2}{8} = \frac{0.020 \cdot 21.33^2}{8} = 1.14 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to spreader beam weight =  $M_{\text{spdr}}$

$$M_{\text{spdr}} = \frac{P_{\text{spdr}} \cdot L_e}{3} = \frac{0.025 \cdot 21.33}{3} = 0.18 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to curb/railing weight =  $M_{\text{barrier}}$

$$M_{\text{barrier}} = \frac{w_{\text{barrier}} \cdot (L_e)^2}{8} = \frac{0.006 \cdot 21.33^2}{8} = 0.34 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to bridge component dead loads =  $M_{\text{dc}}$

$$M_{\text{dc}} = M_{\text{deck}} + M_{\text{spdr}} + M_{\text{barrier}}$$

$$M_{\text{dc}} = 3.30 + 0.18 + 0.34 = 3.82 \text{ kip-ft/ft}$$

Maximum bending moments due to wearing course loads =  $M_{dw}$

$$M_{dw} = M_{ws} + M_{FWC}$$

$$M_{dw} = 2.84 + 1.14 = 3.98 \text{ kip-ft/ft}$$

**[3.6.1.2]**

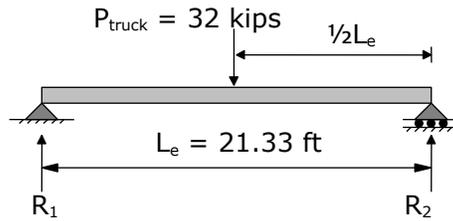
**C. Live Load Moments per Lane (12 ft)**

The live load bending moment will be calculated per lane (12 ft) and later converted to a per unit strip (1 ft) format.

**[3.6.1.2.2]**

**1. Design Truck Axle Loads**

Point load of design truck axle =  $P_{truck} = 32$  kips



**[AISC 14<sup>th</sup> p. 3-215]**

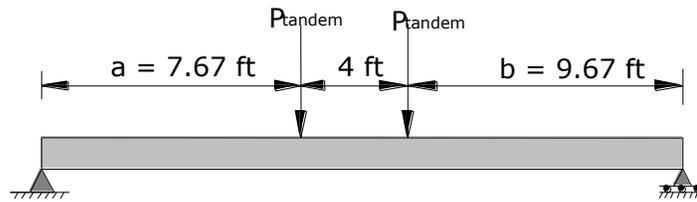
Maximum bending moment due to design truck axle load =  $M_{truck}$

$$M_{truck} = \frac{P_{truck} \cdot L_e}{4} = \frac{32 \cdot 21.33}{4} = 170.64 \frac{\text{kip-ft}}{\text{lane}}$$

**[3.6.1.2.3]**

**2. Design Tandem Axle Loads**

Point load of design tandem axle =  $P_{tandem} = 25$  kips



**[AISC 14<sup>th</sup> p. 3-228]**

Maximum bending moment due to design tandem axle loads =  $M_{tandem}$

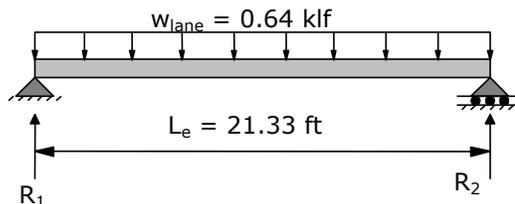
$$M_{tandem} = 12.5 \cdot L_e - 50 + \frac{50}{L_e} = 12.5 \cdot 21.33 - 50 + \frac{50}{21.33} = 218.97 \frac{\text{kip-ft}}{\text{lane}}$$

This moment is assumed to occur at the span 0.50 point.

**[3.6.1.2.4]**

**3. Design Lane Loads**

Uniform design lane load =  $w_{lane} = 0.64$  klf



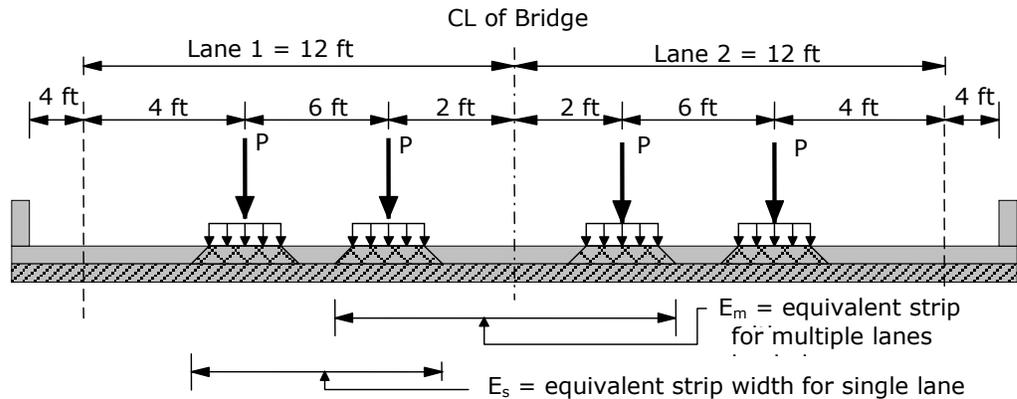
Maximum bending moment due to design lane load =  $M_{lane}$

$$M_{lane} = \frac{W_{lane} \cdot (L_e)^2}{8} = \frac{0.64 \cdot 21.33^2}{8} = 36.40 \frac{\text{kip-ft}}{\text{lane}}$$

[4.6.2.3]

**D. Live Load Equivalent Lane Strip Width**

The live load bending moments, calculated above, will now be distributed over the transverse equivalent lane distance ( $E_m$  or  $E_s$ ).



Physical edge-to-edge bridge deck width =  $W = b_{bridge} = 34.0 \text{ ft}$

$$L_e = 21.33 \text{ ft} \leq 60 \text{ ft}$$

Therefore, modified span length =  $L_1 = L_e = 21.33 \text{ ft}$

[3.6.1.1.1]

Number of traffic lanes on the deck =  $N_L$

$$N_L = \frac{b_{rd}}{12 \frac{\text{ft}}{\text{lane}}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

**1. Single Lane Loaded**

[Eqn. 4.6.2.3-1]

$W = b_{bridge} = 34.0 \text{ ft} > 30 \text{ ft}$

Therefore, the modified edge-to-edge bridge width for single lane load case =  $W_1 = 30 \text{ ft}$

Equivalent lane strip width for single lane loaded =  $E_s$

$$E_s = 10 + 5.0 \cdot \sqrt{L_1 \cdot W_1} = 10 + 5 \cdot \sqrt{21.33 \cdot 30} = 136.48 \frac{\text{in}}{\text{lane}} = 11.37 \frac{\text{ft}}{\text{lane}}$$

[Eqn. 4.6.2.3-2]

**2. Multiple Lanes Loaded**

$W = b_{bridge} = 34.0 \text{ ft} \leq 60 \text{ ft}$

Therefore, the modified edge-to-edge bridge width for multiple lanes loaded case =  $W_1 = 34.0 \text{ ft}$ .

Equivalent lane strip width for multiple lanes loaded =  $E_m$  = lesser of

$$E_m = 12 \cdot \frac{W}{N_L} = 12 \cdot \frac{34.0}{2} = 204.0 \frac{\text{in}}{\text{lane}} = 17.0 \frac{\text{ft}}{\text{lane}}$$

OR

$$E_m = 84 + 1.44 \sqrt{L_1 \cdot W_1} = 84 + 1.44 \sqrt{21.33 \cdot 34} = 122.78 \frac{\text{in}}{\text{lane}} = 10.23 \frac{\text{ft}}{\text{lane}}$$

Use  $E_m = 122.78 \text{ in/lane} = 10.23 \text{ ft/lane}$

## E. Modification of Live Load Bending Moments

[3.6.1.1.2, 4.6.2.3]

### 1. Multiple Presence Factors

The multiple presence factors cannot be used in conjunction with the equivalent lane strip widths of Article 4.6.2.3. The multiple presence factors have already been included in these equations.

[C3.6.1.1.2]

This design example is for an unspecified ADTT, although as stated in Article 8.2.1 of this manual, AASHTO LRFD recommends limitations on the use of wood deck types based on ADTT. If these recommendations are adhered to, AASHTO LRFD also allows reduction of force effects based on ADTT because the multiple presence factors were developed on the basis of an ADTT of 5000 trucks in one direction. A reduction of 5% to 10% may be applied if the ADTT is expected to be below specified limits during the life of the bridge. If the ADTT level is confirmed, the reduction may be applied subject to the judgment of the designer and approved by the State Bridge Design Engineer.

## 2. Convert Live Load Bending Moments to per Unit Strip

### a. Single Lane Loaded Case

$$E_s = 11.37 \text{ ft/lane}$$

Maximum moment from one lane of design truck loads =  $M_{\text{truck}(s)}$

$$M_{\text{truck}(s)} = M_{\text{truck}} \cdot \frac{1}{E_s} = 170.64 \cdot \frac{1}{11.37} = 15.01 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum moment from one lane of design tandem loads =  $M_{\text{tandem}(s)}$

$$M_{\text{tandem}(s)} = M_{\text{tandem}} \cdot \frac{1}{E_s} = 218.97 \cdot \frac{1}{11.37} = 19.26 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum moment from one design lane load case =  $M_{\text{lane}(s)}$

$$M_{\text{lane}(s)} = M_{\text{lane}} \cdot \frac{1}{E_s} = 36.4 \cdot \frac{1}{11.37} = 3.20 \frac{\text{kip-ft}}{\text{ft}}$$

### b. Multiple Lanes Loaded Case

$$E_m = 10.23 \text{ ft/lane}$$

Maximum moment from two lanes of design truck loads =  $M_{\text{truck}(m)}$

$$M_{\text{truck}(m)} = M_{\text{truck}} \cdot \frac{1}{E_m} = 170.64 \cdot \frac{1}{10.23} = 16.68 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum moment from two lanes of design tandem loads =  $M_{\text{tandem}(m)}$

$$M_{\text{tandem}(m)} = M_{\text{tandem}} \cdot \frac{1}{E_m} = 218.97 \cdot \frac{1}{10.23} = 21.40 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum moment from two design lane loads =  $M_{\text{lane}(m)}$

$$M_{\text{lane}(m)} = M_{\text{lane}} \cdot \frac{1}{E_m} = 36.4 \cdot \frac{1}{10.23} = 3.56 \frac{\text{kip-ft}}{\text{ft}}$$

## F. Summary of Unfactored Dead and Live Load Bending Moments for a Unit Strip (1 ft) of Deck

**Table 8.7.1.1 - Applied Bending Moments**

Unfactored Load Case	Maximum Positive Bending Moment (kip-ft/ft)
Dead Loads	
Bridge Components ( $M_{dc}$ )	3.82
Bridge Wearing Surface ( $M_{dw}$ )	3.98
Live Loads (Single Lane Loaded)	
Design Truck	15.01
Design Tandem	19.26
Design Lane	3.20
Live Loads (Two Lanes Loaded)	
Design Truck	16.68
Design Tandem	21.40
Design Lane	3.56

## G. Factored Bending Moment per Unit Strip (1 ft)

### 1. Load Modifiers

Standard MnDOT load modifiers are summarized in Table 3.2.1. of this manual.

For timber bridges  $\eta_D = 1.0$ . MnDOT considers spike laminated decks to have a conventional level of redundancy and uses  $\eta_R = 1.0$ . This example bridge is assumed to have a design ADT of over 500 for  $\eta_I = 1.0$ .

**[1.3.2]**

Therefore, importance, redundancy, and ductility factors =  $\eta = 1.0$

## 2. Strength I Limit State Load Factors

[3.4.1]

Use the Strength I Limit State to determine the required resistance for the deck panels.

[3.6.2.3]

Impact factor need not be applied to wood components.

[4.6.2.3]

Skew factor (bridge is not skewed) =  $r = 1.0$

Specific Strength I Limit State load factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis showed that the tandem axle load controls the bending moment of the deck panels. Additionally, the previous results indicate that the live loads per unit strip are largest for the two lanes loaded case. Therefore, use the two lanes loaded case of the tandem axle loads with the uniform lane load in determining the critical live load bending moment acting on the deck panels.

[Tables 3.4.1-1  
and 3.4.1-2]

## 3. Strength I Limit State Bending Moment per Unit Strip (1 ft)

Factored bending moment for two lanes loaded case =  $M_{u(m)}$

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot [M_{tandem(m)} + M_{lane(m)}]]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot 3.82 + 1.50 \cdot 3.98 + 1.75 \cdot 1.0 \cdot [(21.40 + 3.56)]] = 54.43 \frac{\text{kip-ft}}{\text{ft}}$$

**Check Flexural  
Resistance of Deck  
Panel**

## A. Factored Flexural Resistance

The factored bending moment ( $M_{u(m)}$ ) is the required flexural resistance of the deck that needs to be compared with the actual factored flexural resistance of the deck panel ( $M_R$ ).

[8.6.2]

For a rectangular wood section  $M_R = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$ .

[8.5.2.2]

### 1. Resistance Factors

Flexural resistance factor =  $\phi_f = 0.85$

Compression perpendicular to grain resistance factor =  $\phi_{cperp} = 0.90$

[8.6.2]

### 2. Stability Factor

Stability factor for sawn dimension lumber in flexure =  $C_L$

Laminated deck planks are fully braced.  $C_L = 1.0$

[8.4.4.4]

[Table 8.4.4.4-1]

### 3. Adjustment Factors for Reference Design Value

Size effect factor for sawn dimension lumber in flexure =  $C_F$

$$d_{lam} = 14 \text{ in}$$

$$b_{lam} = 4 \text{ in}$$

$$C_F = 1.00$$

- [8.4.4.2]** Format conversion factor for component in flexure =  $C_{KF}$   
 $C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$
- [8.4.4.3]** Wet service factor for sawn dimension lumber in flexure =  $C_M$   
**[Table 8.4.4.3-1]** Check  $F_{b0} \cdot C_F$ :  $1.00 \cdot 1.0 = 1.0 \leq 1.15$   
 $C_M = 1.00$
- [8.4.4.7]** Incising factor for dimension lumber in flexure =  $C_i$   
 Douglas Fir-Larch requires incising for penetration of treatment.  
**[Table 8.4.4.7-1]**  $C_i = 0.80$
- [8.4.4.8]** Deck factor for a spike-laminated deck in flexure =  $C_d$   
**[Table 8.4.4.8-1]**  $C_d = 1.15$
- [8.4.4.9]** Time effect factor for Strength I Limit State =  $C_\lambda$   
**[Table 8.4.4.9-1]**  $C_\lambda = 0.80$
- [Eqn. 8.4.4.1-1]** Adjusted design value =  $F_b = F_{b0} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_i \cdot C_d \cdot C_\lambda$   
 $F_b = 1.00 \cdot 2.94 \cdot 1.00 \cdot 1.00 \cdot 0.80 \cdot 1.15 \cdot 0.80 = 2.16 \text{ ksi}$

#### 4. Required Section Modulus

The section modulus is dependent on the deck panel depth. The section modulus is used in Part B to solve for the deck panel depth.

#### B. Required Deck Panel Depth

Required deck flexural resistance =  $M_{n(\text{req})}$

For the deck panel depth to meet Strength I Limit State,  $M_r$  must equal (or exceed)  $M_{u(m)}$ , where  $M_r = \phi M_{n(\text{req})}$ . Therefore, set  $\phi M_{n(\text{req})} = M_{u(m)}$ .

$$M_{n(\text{req})} = \frac{M_{u(m)}}{\phi_f} = \frac{54.43}{0.85} = 64.04 \text{ kip} \cdot \text{ft}$$

Required section modulus of one foot of deck width =  $S_{\text{req}}$

Required depth of deck laminates (panel) =  $d_{\text{req}}$

$$S_{\text{req}} = \frac{12 \cdot d_{\text{req}}^2}{6}$$

$$M_{n(\text{req})} = F_b \cdot S_{\text{req}} \cdot C_L, \text{ with } C_L = 1.0$$

Substituting terms gives

$$d_{\text{req}} = \sqrt{\frac{6 \cdot M_{n(\text{req})}}{12 \cdot F_b \cdot C_L}} = \sqrt{\frac{6 \cdot 64.04 \cdot 12}{12 \cdot 2.16 \cdot 1.0}} = 13.34 \text{ in} \leq 14.0 \text{ in}$$

OK

The required deck panel depth (13.34 inches) indicates that the originally assumed deck depth (14 inches) can be used. However, it is not uncommon that a deeper section could be required to satisfy the deflection limit, so that is checked next.

**Investigate  
Deflection  
Requirements**

[8.5.1]

[2.5.2.6.2]

[3.6.1.3.2]

[9.9.3.3]

**A. Deck Live Load Deflection with Current Deck Parameters**

The midspan deflections are estimated with the design truck or 25% of the design truck applied in conjunction with the design lane load.

Deflections are to be calculated using Service I Limit State.

Design for deflections using a per foot width approach. With all design lanes loaded, it is allowed to assume all supporting components deflect equally for straight girder systems. This approach can be used on a spike laminated deck with spreader beams meeting the requirements of AASHTO LRFD.

[2.5.2.6.2]

[C2.5.2.6.2]

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

**1. Deck Stiffness**

Moment of inertia of one foot width of deck panels =  $I_{prov}$

$$I_{prov} = \frac{1}{12} \cdot b \cdot d_{lam}^3 = \frac{1}{12} \cdot 12 \cdot (14)^3 = 2744 \text{ in}^4$$

Adjusted deck panel modulus of elasticity =  $E$

[8.4.4.3]

[Table 8.4.4.3-1]

Wet service factor, modulus of elasticity of sawn dimension lumber =  $C_M$

$$C_M = 0.90$$

[8.4.4.7]

[Table 8.4.4.7-1]

Incising factor, modulus of elasticity of sawn dimension lumber =  $C_i$

Douglas Fir-Larch requires incising for penetration of treatment.

$$C_i = 0.95$$

[Eqn. 8.4.4.1-6]

Adjusted design value =  $E = E_o \cdot C_M \cdot C_i$

$$E = 1700 \text{ ksi} \cdot 0.90 \cdot 0.95 = 1453.5 \text{ ksi}$$

**2. Loads per Unit Strip Width (1 ft)**

Design truck load used for deflection calculations =  $P_{\Delta truck}$

$$P_{\Delta truck} = [2 \text{ lanes of load}] / b_{bridge}$$

$$P_{\Delta truck} = [2 \cdot 32 \text{ kips}] / 34.0 \text{ ft} = 1.882 \text{ kips/ft}$$

Design lane load used for deflection calculations =  $w_{\Delta lane}$

$$W_{\Delta \text{lane}} = 2 \text{ lanes of load} / b_{\text{bridge}} = 2 \cdot 0.64 \text{ klf} / 34.0 \text{ ft} \\ = 0.038 \text{ klf/ft}$$

**[3.6.1.3.2]**

**[AISC 14<sup>th</sup> p. 3-213,  
3-215]**

**3. Live Load Deflection Calculations**

Deflection at deck midspan due to the design truck load =  $\Delta_{\text{truck}}$

$$\Delta_{\text{truck}} = \frac{P_{\text{truck}} \cdot L_e^3}{48 \cdot E \cdot I_{\text{prov}}} = \frac{1.882 \cdot (21.33 \cdot 12)^3}{48 \cdot 1453.5 \cdot 2744} = 0.16 \text{ in}$$

Deflection at deck midspan due to the design lane load =  $\Delta_{\text{lane}}$

$$\Delta_{\text{lane}} = \frac{5 \cdot W_{\text{lane}} \cdot L_e^4}{384 \cdot E \cdot I_{\text{prov}}} = \frac{5 \cdot \frac{0.038}{12} \cdot (21.33 \cdot 12)^4}{384 \cdot 1453.5 \cdot 2744} = 0.04 \text{ in}$$

Deflection at deck midspan due to a combination of truck (25%) and design lane loads =  $\Delta_{\text{combined}}$

$$\Delta_{\text{combined}} = 0.25 \cdot \Delta_{\text{truck}} + \Delta_{\text{lane}} = (0.25 \cdot 0.16) + 0.04$$

$$\Delta_{\text{combined}} = 0.08 \text{ in} \leq \Delta_{\text{truck}} = 0.16 \text{ in}$$

Therefore, the maximum deflection between the combination load deflection and the truck load deflection =  $\Delta = \Delta_{\text{truck}} = 0.16 \text{ in}$ .

**[2.5.2.6.2]**

Live load deflection limit at deck midspan =  $\Delta_{\text{max}}$

$$\Delta_{\text{max}} = L_e / 425 = 21.33 / 425 = 0.0502 \text{ ft} = 0.60 \text{ in}$$

$$\Delta = 0.16 \text{ in} \leq \Delta_{\text{max}} = 0.60 \text{ in}$$

OK

The initial 14-inch deck panel depth and grade are adequate for deflection.

**Check Shear  
Resistance  
Of Deck Panel  
[8.7, 9.9.3.2]**

In longitudinal decks, maximum shear shall be computed in accordance with the provisions of AASHTO LRFD Article 8.7. For this example, shear loading is not close to governing the design of the deck panel and so the calculation is not shown here. Shear check for a transverse deck is shown in the glulam beam with transverse deck design example (Article 8.7.4).

**Investigate  
Spreader Beam  
Requirements  
[9.9.6.3]  
[9.9.4.3]**

**A. Spreader Beam Parameters**

A spreader beam is required to satisfy the AASHTO definition of interconnected spike laminated panels.

$$\text{Maximum spreader beam spacing} = s_{\text{max}} = 8.0 \text{ ft}$$

$$\text{Actual longitudinal spreader beam spacing} = s_{\text{spdr}} = L / 3 \\ = 22 / 3 = 7.33 \text{ ft}$$

$$s_{\text{spdr}} = 7.33 \text{ ft} \leq s_{\text{max}} = 8.0 \text{ ft}$$

OK

$$\text{Minimum allowed rigidity of the spreader beams} = EI_{\text{min}} = 80,000 \text{ kip}\cdot\text{in}^2$$

The spreader beams shall be attached to each deck panel near the panel edges and at intervals less than or equal to 15 inches. The spreader beams also reduce the relative panel deflection, thus aiding to decrease wearing surface cracking. If bituminous maintenance is a concern, exceeding the minimum criteria for spacing (adding more spreader beams) may increase wearing surface expected life.

Required moment of inertia of spreader beams to accommodate the specified rigidity for a given species and grade of wood =  $I_{\text{min}}$ .

For Douglas Fir-Larch No. 1 Beams & Stringers (B & S),  $E_o = 1600 \text{ ksi}$

Adjusted spreader beam modulus of elasticity =  $E$

**[8.4.4.3]**

**[Table 8.4.4.3-1]**

Wet service factor for modulus of elasticity of B & S timber =  $C_M$

For nominal thickness > 4.0 in,  $C_M = 1.0$

**[Eqn. 8.4.4.1-6]**

Adjusted design value =  $E = E_o \cdot C_M$

$$E = 1600 \cdot 1.0 = 1600 \text{ ksi}$$

$$I_{\text{min}} = \frac{80,000}{E} = \frac{80,000}{1600} = 50.0 \text{ in}^4$$

Find required depth of spreader beam =  $d_{\text{min}}$

$$I_{\text{min}} = \frac{1}{12} \cdot b_{\text{spdr}} \cdot d_{\text{min}}^3$$

$$d_{\text{min}} = \sqrt[3]{\frac{12 \cdot I_{\text{min}}}{b_{\text{spdr}}}} = \sqrt[3]{\frac{12 \cdot 50.0}{6}} = 4.64 \text{ in} \leq d_{\text{spdr}} = 12 \text{ in} \quad (\text{OK})$$

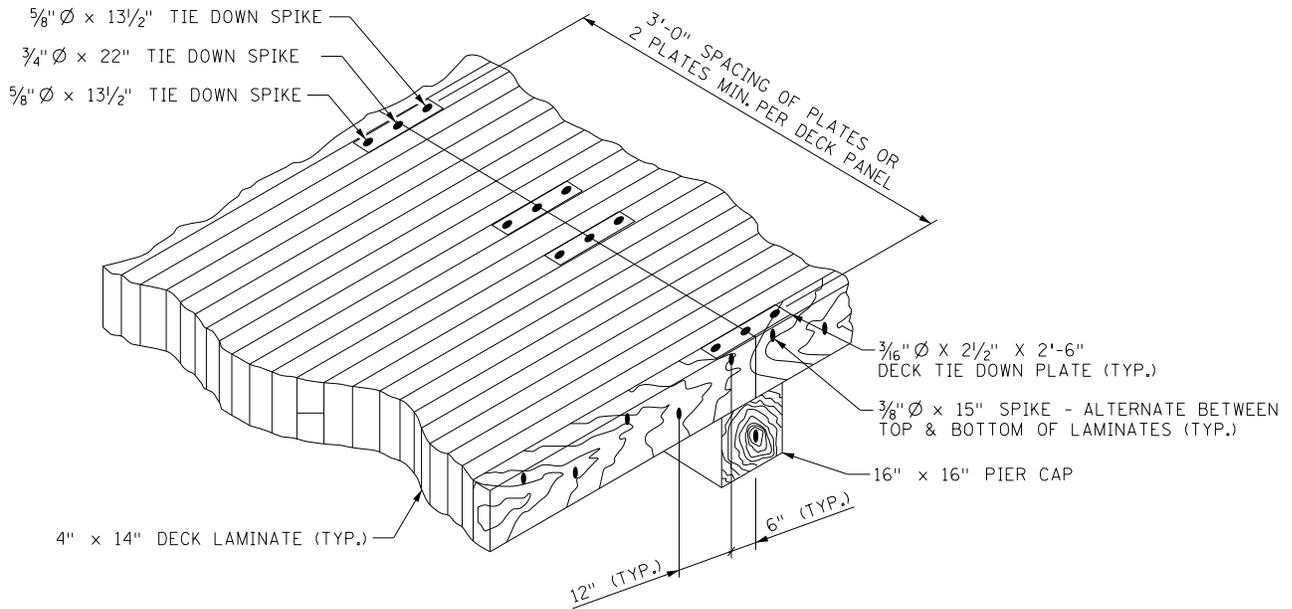
As described in Article 8.2.3 of this manual, MnDOT standard practice is to use 6 in X 12 in spreader beams, which exceed the specified minimum criteria.

**[9.9.6.1]**

**B. Spike Lamination Deck Pattern**

Spike-laminated decks shall consist of a series of lumber laminations that are placed edgewise between supports and spiked together on their wide face with deformed spikes of sufficient length to fully penetrate four laminations. The spikes shall be placed in lead holes that are bored through pairs of laminations at each end and at intervals not greater than 12.0 inches in an alternating pattern near the top and bottom of the laminations.

Laminations shall not be butt spliced within their unsupported length.



\*Typical each deck Tie-down

**Figure 8.7.1.2 – Longitudinal Timber Deck to Cap Connections**

[9.9.6.2, 9.9.4.2]

**C. Deck Tie-Downs**

Typically, MnDOT uses  $\frac{5}{8}$  inch diameter spikes to attach the metal tie-down plates (brackets) to the deck panels, and  $\frac{3}{4}$  inch diameter spikes are used to connect the plates to the pile cap. The plates are typically  $\frac{3}{16}$  inch thick by  $2\frac{1}{2}$  inches wide X 2'-6" long. These plates can be spaced at 3 feet maximum intervals transversely over the pile cap as specified for stress laminated decks or a minimum of two plates per deck panel, with the latter being more typical of MnDOT designs.

**Investigate Bearing Strength Requirements**

**A. Maximum Support Reactions per Unit Strip (1 ft)**

**1. Live Load Reactions**

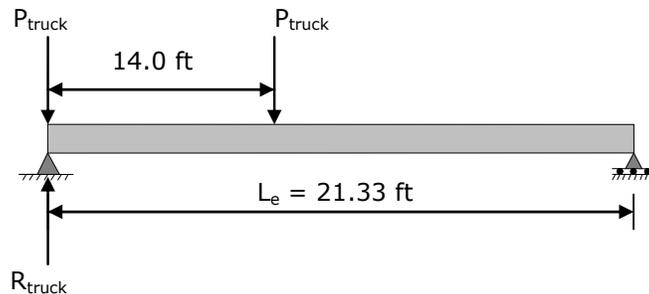
The maximum live load reactions need to be calculated. The design truck and tandem axle loads have been oriented to produce the greatest reaction at the pile cap. The design truck, tandem, and lane reactions are assumed to be uniformly distributed over the equivalent live load strip width ( $E_s$  or  $E_m$ ).

**a. Multiple Lanes Loaded**

The calculations below only consider the multiple lanes loaded case. Because the equivalent lane strip width for multiple lanes is less than that

for the single lane loaded case ( $E_m < E_s$ ), there is more force per transverse foot for the multiple lane load case.

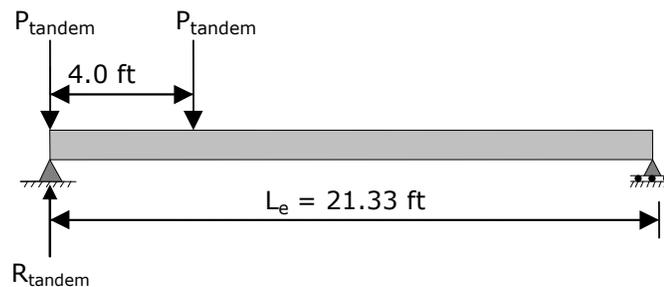
Maximum pile cap reaction due to the design truck loads =  $R_{truck}$



$$R_{truck} = \left[ P_{truck} + P_{truck} \cdot \frac{(L_e - 14)}{L_e} \right] \cdot \frac{1}{E_m}$$

$$R_{truck} = \left[ 32 + 32 \cdot \frac{(21.33 - 14)}{21.33} \right] \cdot \frac{1}{10.23} = 4.202 \frac{\text{kips}}{\text{ft}}$$

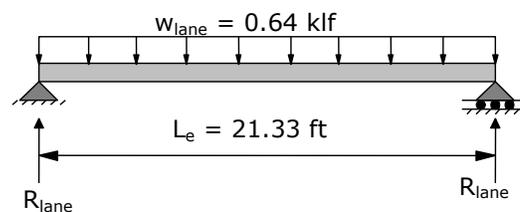
Maximum pile cap reaction due to the design tandem loads =  $R_{tandem}$



$$R_{tandem} = \left[ P_{tandem} + P_{tandem} \cdot \frac{(L_e - 4)}{L_e} \right] \cdot \frac{1}{E_m}$$

$$R_{tandem} = \left[ 25 + 25 \cdot \frac{(21.33 - 4)}{21.33} \right] \cdot \frac{1}{10.23} = 4.429 \frac{\text{kips}}{\text{ft}}$$

Maximum pile cap reaction due to the design lane load =  $R_{lane}$



$$R_{lane} = \left[ \frac{W_{lane} \cdot L_e}{2} \right] \cdot \frac{1}{E_m} = \left[ \frac{0.64 \cdot 21.33}{2} \right] \cdot \frac{1}{10.23} = 0.667 \frac{\text{kips}}{\text{ft}}$$

## 2. Dead Load Reactions

Maximum reaction on pile cap due to the deck weight =  $R_{deck}$

$$R_{deck} = \frac{W_{deck} \cdot L_e}{2} = \frac{0.058 \cdot 21.33}{2} = 0.622 \frac{\text{kip}}{\text{ft}}$$

Maximum reaction on pile cap due to the wearing surface weight =  $R_{ws}$

$$R_{ws} = \frac{W_{ws} \cdot L_e}{2} = \frac{0.050 \cdot 21.33}{2} = 0.533 \frac{\text{kips}}{\text{ft}}$$

Maximum reaction on cap due to future wearing surface weight =  $R_{FWC}$

$$R_{FWC} = \frac{W_{FWC} \cdot L_e}{2} = \frac{0.020 \cdot 21.33}{2} = 0.213 \frac{\text{kips}}{\text{ft}}$$

Maximum pile cap reaction due to spreader beam =  $R_{spdr}$

$$R_{spdr} = 0.025 \text{ kips/ft}$$

Maximum reaction on pile cap due to the curb/railing weight =  $R_{barrier}$

$$R_{barrier} = \frac{W_{barrier} \cdot L_e}{2} = \frac{0.006 \cdot 21.33}{2} = 0.064 \frac{\text{kips}}{\text{ft}}$$

Maximum reaction on pile cap due to the component dead loads =  $R_{dc}$

$$R_{dc} = R_{deck} + R_{spdr} + R_{barrier}$$

$$R_{dc} = 0.622 + 0.025 + 0.064 = 0.711 \text{ kips/ft}$$

Maximum reaction on pile cap due to the wearing course =  $R_{dw}$

$$R_{dw} = R_{ws} + R_{FWC}$$

$$R_{dw} = 0.533 + 0.213 = 0.746 \text{ kips/ft}$$

## B. Summary of Unfactored Support Reactions

**Table 8.7.1.2 – Support Reactions**

Unfactored Load Case	Maximum Support Reaction (kips/ft)
Dead Loads	
Bridge Components ( $R_{dc}$ )	0.711
Bridge Wearing Surface ( $R_{dw}$ )	0.746
Live Loads (Two Lanes Loaded)	
Design Truck	4.202*
Design Tandem	4.429*
Design Lane	0.667*

\* Kips per transverse foot of the equivalent lane strip ( $E_m$ )

**C. Strength I Limit State Reaction per Unit Strip (1 ft)****[3.4.1]**

Table 8.7.1.2 shows that the design tandem for the two lanes loaded case controls the reaction of the deck panels. Therefore, the design truck will be neglected for bearing calculations.

Maximum factored reaction when multiple lanes are loaded =  $R_{u(m)}$

**[Tables 3.4.1-1 and 3.4.1-2]**

$$R_{u(m)} = \eta \cdot [1.25 \cdot R_{dc} + 1.50 \cdot R_{dw} + 1.75 \cdot r \cdot (R_{tandem(m)} + R_{lane(m)})]$$

$$R_{u(m)} = 1.0 \cdot [1.25 \cdot 0.711 + 1.50 \cdot 0.746 + 1.75 \cdot 1.0 \cdot (4.429 + 0.667)] = 10.926 \frac{\text{kips}}{\text{ft}}$$

**[8.8.3]****D. Factored Bearing Resistance**

The factored resistance ( $P_r$ ) of a component in compression perpendicular to grain shall be taken as  $P_r = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b$

**[Eqns. 8.8.1-1, 8.8.3-1]****1. Bearing Area**

Width of bearing =  $b_b = 1 \text{ ft} = 12 \text{ in}$  (for unit strip)

Length of bearing =  $L_b = \frac{1}{2} \cdot b_{cap} = \frac{1}{2} \cdot 16 = 8 \text{ in}$

Provided bearing area =  $A_b = b_b \cdot L_b = 12 \cdot 8 = 96 \text{ in}^2$

**[Table 8.8.3-1]****2. Bearing Adjustment Factor**

$$L_b = 8 \text{ in} \geq 6 \text{ in}$$

$$C_b = 1.0$$

**3. Adjustment Factors for Reference Design Value****[8.4.4.2]**

Format conversion factor for compression perpendicular to grain =  $C_{KF}$

$$C_{KF} = 2.1/\phi_{cperp} = 2.1/0.90 = 2.33$$

**[8.4.4.3]**

Wet Service factor for sawn dimension lumber =  $C_M$

**[Table 8.4.4.3-1]**

$$C_M = 0.67$$

**[8.4.4.7]**

Incising Factor for sawn dimension lumber in compression perpendicular to grain =  $C_i$

**[Table 8.4.4.7-1]**

$$C_i = 1.00$$

**[8.4.4.9]**

Time effect factor for Strength I limit state =  $C_\lambda$

**[Table 8.4.4.9-1]**

$$C_\lambda = 0.80$$

Adjusted design value in compression perpendicular to grain =  $F_{cp}$

**[Eqn. 8.4.4.1-5]**

$$F_{cp} = F_{cpo} \cdot C_{KF} \cdot C_M \cdot C_i \cdot C_\lambda = 0.625 \cdot 2.33 \cdot 0.67 \cdot 1.00 \cdot 0.80$$

$$F_{cp} = 0.781 \text{ ksi}$$

**4. Bearing Resistance Calculation Check**

Nominal resistance of deck in compression perp. to the grain =  $P_n$

$$P_n = F_{cp} \cdot A_b \cdot C_b = 0.781 \cdot 96 \cdot 1.0 = 75.0 \text{ kips/ft}$$

Per foot of width of bearing, the factored resistance of deck in compression perp. to the grain =  $P_r = \phi P_n$

$$\phi P_n = \phi_{c\text{perp}} \cdot P_n = 0.90 \cdot 75.0 \text{ kips/ft} = 67.5 \text{ kips/ft}$$

$$\phi P_n = 67.5 \text{ kips/ft} \geq R_{u(m)} = 10.9 \text{ kips/ft} \quad \text{OK}$$

There is no need to attach a sill component to the cap for extending the bearing because the given bearing strength is more than adequate.

**Summary of  
Connection Design**

Figure 8.7.1.3 below indicates the position of the spreader beam connections, the ship lap joints (deck panel-to-deck panel connections), and deck panel-to-pile cap tie-down plates. For connections not specified in AASHTO, or for the use of connections that are not in accordance with AASHTO, State Bridge Design Engineer approval is needed.

**[9.9.4.3]**

The maximum spacing of the spreader beam connection bolts is 15 inches, and they shall be placed near the panel edges.

**[8.4.2]**

Minimum fastener and hardware requirements are specified in Section 8 of AASHTO LRFD.

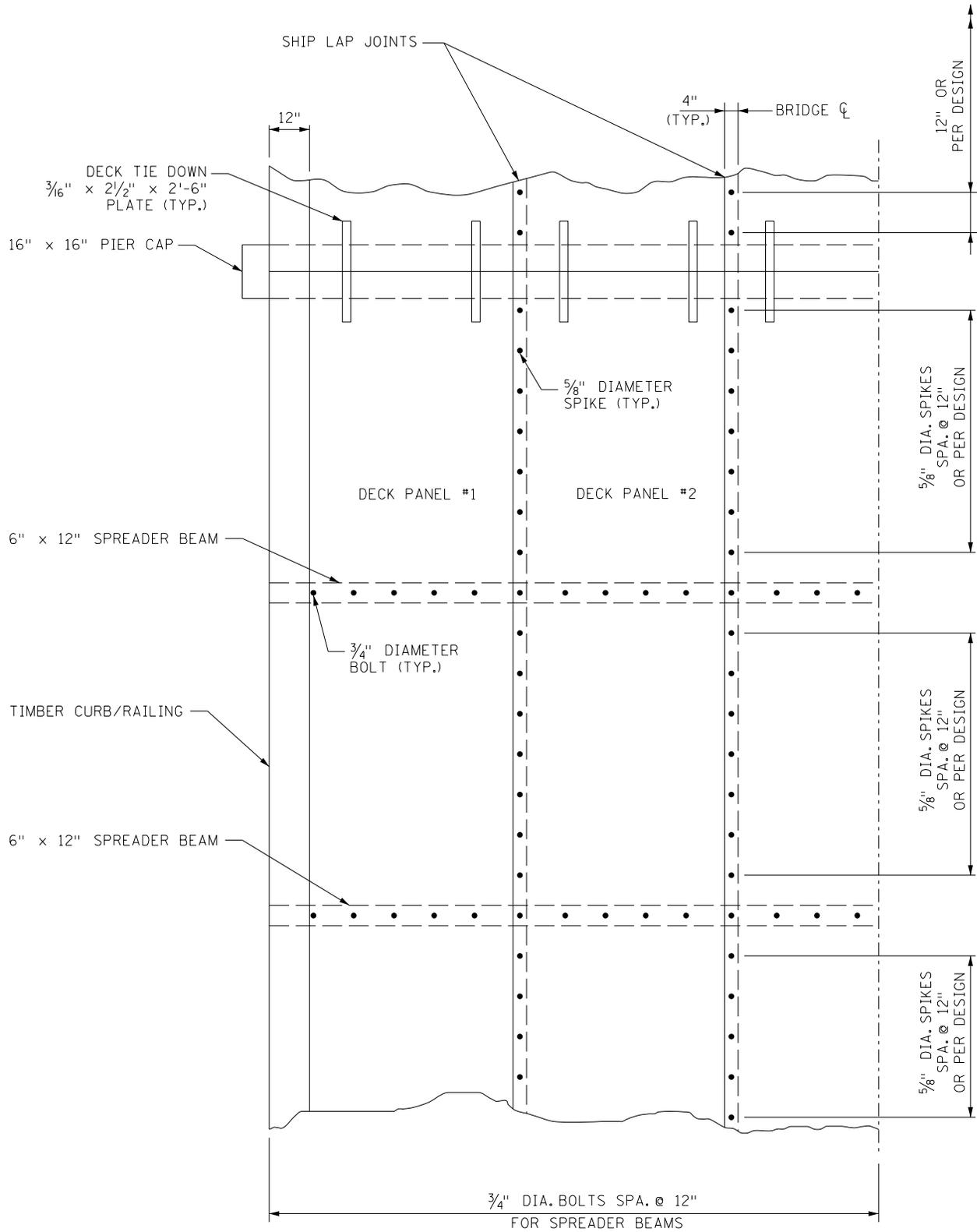


Figure 8.7.1.3 – Longitudinal Timber Deck Partial Plan View

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**8.7.2 Timber Pile  
Cap Design  
Example**

This example demonstrates the design of a typical timber pile cap, which accompanies the Longitudinal Spike Laminated Timber Deck design example in Article 8.7.1. The caps provide bearing support of the longitudinal deck for an intermediate bridge span as previously designed. The bridge contains no longitudinal girders; the dead and live loads are distributed loads along the pile cap. These types of bridges are usually reserved for secondary roads with low truck traffic volumes.

**[Figure 8.3-1]**

**A. Material and Design Parameters**

The dimension annotations used throughout this design example are as follows. The vertical dimension of a member is considered its depth. The transverse and longitudinal dimensions of a member are considered its width and length, respectively. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications*, except for sawn lumber descriptive names. The letter notations will be used in this example (b, d, etc.).

**[8.4.1.1]**

**1. Pile Cap**

Initial timber pile cap width =  $b_{cap} = 16 \text{ in} = 1.33 \text{ ft}$

Initial timber pile cap depth =  $d_{cap} = 16 \text{ in} = 1.33 \text{ ft}$

**[8.4.1.1.2]**

The largest size commonly available for visually-graded Posts and Timbers sawn lumber is 16 in X 16 in. Availability of lengths over 36 feet can possibly be limited, and may require a splice. This example does not require a splice. As stated earlier in Article 8.7.1, the dimensions for the rough sawn caps are used as actual.

**[9.9.8]**

**2. Wearing Course**

Depth of wearing course =  $d_{ws} = 4 \text{ in}$ , which is the average depth taken from the Longitudinal Spike Laminated Timber Deck design example in Article 8.7.1.

**3. Curb and Railing (TL-4 Glulam Timber Rail with Curb)**

Curb and railing components are itemized in the Longitudinal Spike Laminated Timber Deck design example.

The timber barrier design is not a part of this design example.

The maximum spacing for the timber rail posts is 6.25 ft.

**[8.4.1.1]**

**[9.9.2]**

**4. Deck Laminates**

Depth of timber deck panel laminates =  $d_{lam} = 14 \text{ in} = 1.167 \text{ ft}$

Width of timber deck panel laminates =  $b_{lam} = 4 \text{ in} = 0.333 \text{ ft}$

**[8.4.1.1.2]**

Deck panel lumber is designed in Article 8.7.1.

### 5. Piles

Diameter of circular steel shell piles =  $d_{\text{pile}} = 16$  in

Number of piles =  $n_{\text{piles}} = 5$

It is standard MnDOT practice to use equally spaced 16 inch diameter piles for the pile bent piers. Refer to Article 8.4 of this manual for further description.

### 6. Cap Span Lengths

Overall transverse length of pile caps =  $L_{\text{trans}} = 36$  ft

Transverse combined width of deck panels =  $b_{\text{bridge}} = 34.0$  ft

Longitudinal distance between pile cap centerlines =  $L = 22$  ft

Transverse distance between centerlines of piles =  $L_{\text{cap}} = 8.17$  ft

Transverse clear distance between adjacent piles =  $L_{\text{clr}} = 6.83$  ft

The pile cap is not spliced for this design example. When a pile cap is spliced, the splice should be over an interior pile. Refer to Figure 8.7.2.1 below for pile locations. Adjacent spans are  $L = 22$  ft for this example.

### 7. Unit Weights and Moisture Content

Type of pile cap wood material = Douglas Fir-Larch Posts and Timbers (No. 1)

**[Table 3.5.1-1]**

Unit weight of soft wood (Douglas Fir-Larch) =  $\gamma_{\text{DFL}} = 0.050$  kcf

**[MnDOT Table 3.3.1]**

Unit weight of bituminous wearing course =  $\gamma_{\text{WS}} = 0.150$  kcf

**[MnDOT 3.3]**

Standard MnDOT practice is to apply a future wearing course of 20 psf.

**[8.4.1.1.3]**

Moisture content of timber (MC) at the time of installation shall not exceed 19.0%. MnDOT designs for in service wet-use only, which is a MC of greater than 19% for sawn timber.

### 8. Douglas Fir-Larch Posts and Timbers (No. 1) Strength Properties

**[Table 8.4.1.1.4-1]**

Reference Design Value of wood in flexure =  $F_{b0} = 1.20$  ksi

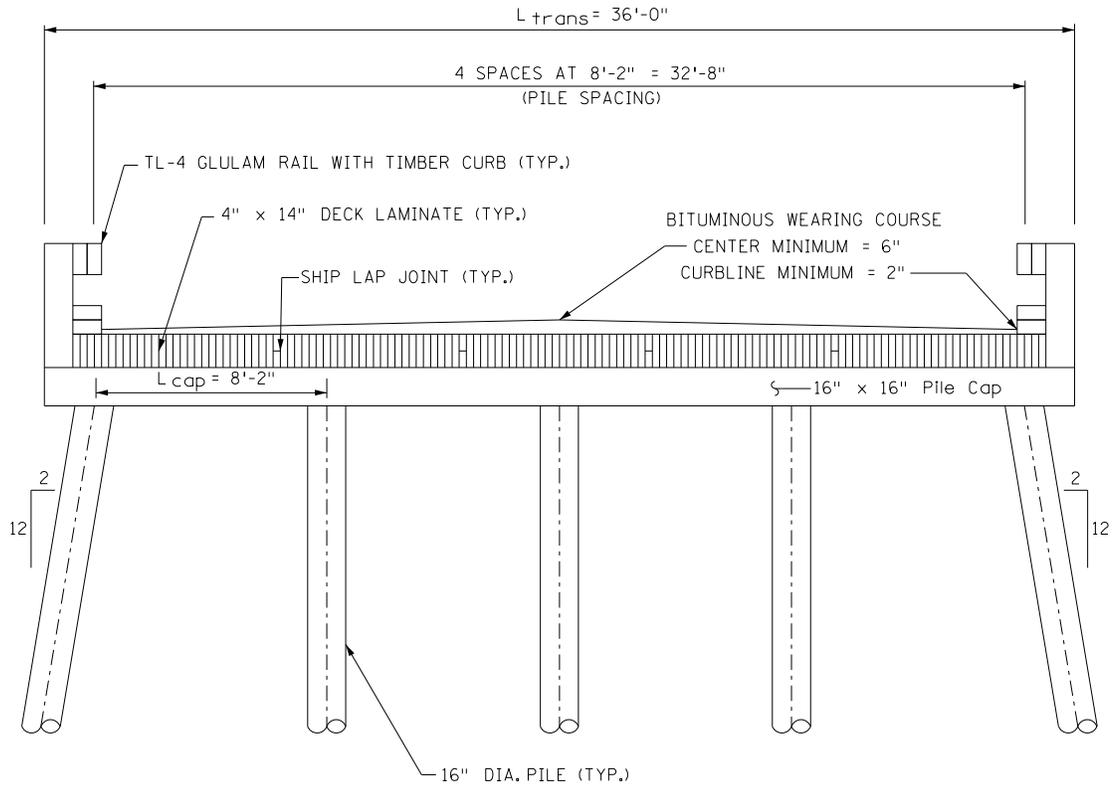
Reference Design Value of wood in horizontal shear =  $F_{v0} = 0.17$  ksi

Reference Design Value of wood in compression perpendicular to grain  
=  $F_{c\text{po}} = 0.625$  ksi

Modulus of elasticity =  $E_o = 1600$  ksi

**Select the Basic Configuration**

The bridge deck consists of 5 interconnected longitudinal deck panels. The deck panels are supported by timber pile caps, which extend the width of the bridge at the piers. See the timber deck example in Article 8.7.1 for details regarding the deck design and connection configurations.



**Figure 8.7.2.1 – Longitudinal Timber Deck on Pier Timber Cap**

**Determine Dead and Live Load Reactions, Shear Forces, and Bending Moments**

**A. Determine Dead Loads**

Dead load units are given in kips per linear foot along the pile cap.

Area of pile cap cross section =  $A_{cap}$   
 $A_{cap} = d_{cap} \cdot b_{cap} = 16 \cdot 16 = 256 \text{ in}^2 = 1.78 \text{ ft}^2$

Linear weight of timber pile cap =  $w_{cap}$   
 $w_{cap} = \gamma_{DFL} \cdot A_{cap} = 0.050 \cdot 1.78 = 0.089 \text{ kips/ft}$

Linear weight of deck panels =  $w_{deck}$   
 $w_{deck} = \gamma_{DFL} \cdot d_{plank} \cdot L = 0.050 \cdot 1.167 \cdot 22 = 1.283 \text{ kips/ft}$

Area of spreader beam =  $A_{spdr}$   
 $A_{spdr} = d_{spdr} \cdot b_{spdr} = 12 \cdot 6 = 72 \text{ in}^2 = 0.5 \text{ ft}^2$

Linear weight of spreader beams =  $w_{spdr}$

$$w_{spdr} = 2 \cdot A_{spdr} \cdot \gamma_{DFL} = 2 \cdot 0.5 \cdot 0.050 = 0.050 \text{ kips/ft}$$

Volume of curb/railing components per longitudinal foot of bridge length  
 $= v_{barrier} = 1.955 \text{ ft}^3/\text{ft}$  (from previous example)

Weight of timber barrier per longitudinal foot of bridge length =  $w_{barrier}$

$$w_{barrier} = \left[ \frac{(2 \cdot \gamma_{DFL} \cdot v_{barrier})}{b_{bridge}} \right] \cdot L = \left[ \frac{(2 \cdot 0.050 \cdot 1.955)}{34.0} \right] \cdot 22 = 0.127 \frac{\text{kips}}{\text{ft}}$$

This linear load assumes that the barrier weight acts uniformly over the entire deck width.

Linear weight of bituminous wearing course =  $w_{ws}$

$$w_{ws} = \gamma_{ws} \cdot d_{ws} \cdot L = 0.150 \cdot 4.0 \cdot (1/12) \cdot 22 = 1.100 \text{ kips/ft}$$

Linear weight of future wearing course =  $w_{FWC}$

$$w_{FWC} = 0.020 \cdot L = 0.020 \cdot 22 = 0.440 \text{ kips/ft}$$

Total linear dead load of components acting along the pile cap =  $w_{dc}$

$$w_{dc} = w_{cap} + w_{deck} + w_{spdr} + w_{barrier}$$

$$w_{dc} = 0.089 + 1.283 + 0.050 + 0.127 = 1.549 \frac{\text{kips}}{\text{ft}}$$

Linear dead load of wearing course acting along the pile cap =  $w_{dw}$

$$w_{dw} = w_{ws} + w_{FWC}$$

$$w_{dw} = 1.100 + 0.440 = 1.540 \frac{\text{kips}}{\text{ft}}$$

## B. Cap Spans and Structural Analysis Models

The pile cap is made up of a four span continuous beam. For simplification, conservative modeling assumptions can be made.

### 1. Analysis Models

In determining the maximum member forces, MnDOT uses a variation of beam models as follows:

- 1) The maximum shear forces and reactions are determined by modeling the pile cap as a continuous beam on pinned supports. Moving live loads are then placed at various locations along the span, to produce the maximum shear and reactions. This method of analysis allows the effects of adjacent spans to be investigated.
- 2) The maximum positive bending moments (tension on pile cap bottom) are determined by considering the pile cap as a single simply-supported span between piles.

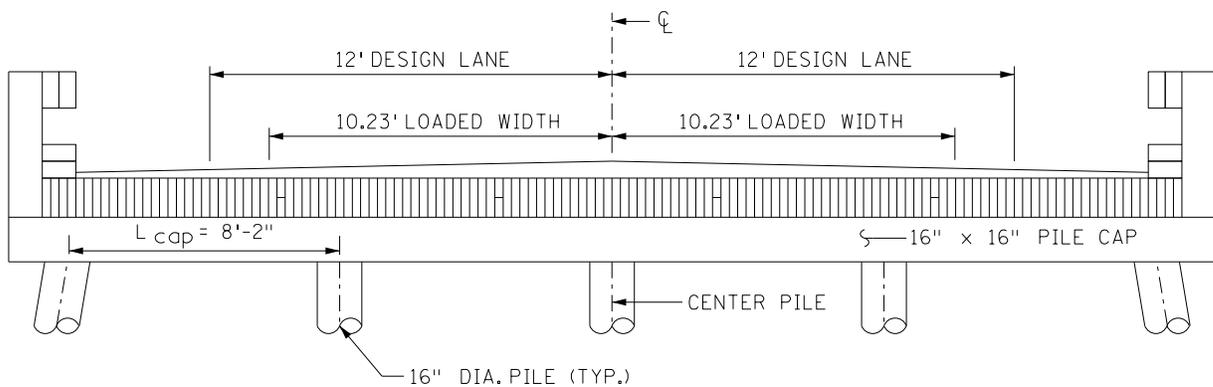
- 3) The maximum negative bending moments (tension on pile cap top) are determined by considering the pile cap as a single fixed-fixed span between piles, with fixed ends.

The dead and live load shear, reactions, and bending moment results can be determined using a basic structural analysis computer program, or using the standard beam formulas found in AISC 14<sup>th</sup> Edition LRFD Manual. The results are summarized in Table 8.7.2.1. The HL-93 reactions for the longitudinal deck are based on Table 3.4.1.1 of this manual in Section 3, for simplicity (except for the lane load). However, for longer spans, the adjacent spans need to be considered in figuring the truck reaction because the third axle will have an increased load effect.

**[3.6.1.3]**

Both the design lanes and 10.0 ft loaded width in each lane shall be positioned to produce extreme force effects. For this timber slab span, the live load is distributed over the equivalent strip widths for a single lane case or multiple lanes case that were calculated in Article 8.7.1. Only one span on the cap and approximately one third of the adjacent span for the single lane case is loaded and so will not control the design of the cap.

For the two lane case the design lanes are side by side, one on each side of the center pile. The loaded width in both design lanes is placed adjacent to the inside of the design lane above the center pile. This position of the design lanes and loaded width will create the largest force effects in the cap. To simplify the calculations of the maximum reactions and shears, it is conservatively assumed that only the two adjacent cap spans are loaded with the distributed live load.



**Figure 8.7.2.2 – Live Load Position for Cap Analysis**

### C. Summary of Maximum Shear Force, Reaction and Bending Moment Results

**Table 8.7.2.1**

Unfactored Load Case	Maximum Positive Bending Moment (kip-ft)	Maximum Negative Bending Moment (kip-ft)	Maximum Shear Force (kips)	Maximum Support Reaction (kips)
Component Dead Load (DC)	12.92	8.62	7.91	15.82
Wearing Course Dead Load (DW)	12.85	8.57	7.86	15.73
Multiple Lanes Loaded				
Design Truck	35.59	23.73	21.78	43.57
Design Tandem	37.07	24.71	22.69	45.37
Design Lane	11.48	7.66	7.03	14.06

### D. Factored Bending Moment in Cap

#### 1. Load Modifiers

Basis for the load modifiers is similar to example 8.7.1.

Importance, redundancy, and ductility factors =  $\eta = 1.0$

[1.3.2]

#### 2. Strength I Limit State Load Factors

Use the Strength I Limit State to determine the required flexural resistance of the pile cap.

[3.6.2.3]

Impact factor need not be applied to wood components.

[4.6.2.3]

Skew factor (bridge is not skewed) =  $r = 1.0$

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results (Table 8.7.2.1) indicate that multiple lanes loaded with the design tandem and lane loads control for flexure.

#### 3. Strength I Limit State Positive Moment

Positive (tension on pile cap bottom) factored bending moment due to multiple lanes loaded case =  $M_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot (M_{\text{tandem}(m)} + M_{\text{lane}(m)})]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot (12.92) + 1.50 \cdot (12.85) + 1.75 \cdot 1.0 \cdot (37.07 + 11.48)]$$

$$M_{u(m)} = 120.39 \text{ kip} - \text{ft}$$

**Check Flexural Resistance of Cap**

### A. Factored Flexural Resistance

The factored bending moment ( $M_{u(m)}$ ) is the required flexural resistance of the cap that needs to be compared with the actual factored flexural resistance of the cap ( $M_R$ ).

[8.6.2]

For a rectangular wood section  $M_R = \phi_f \cdot F_b \cdot S_{\text{prov}} \cdot C_L = M_{R(\text{prov})}$

Because caps are supplied in standard sizes and the dimensions are known,  $M_R$  is calculated as  $M_{R(\text{prov})}$ .

[8.5.2.2]

#### 1. Resistance Factor

Flexural resistance factor =  $\phi_f = 0.85$

#### 2. Section Modulus

The section modulus is dependent on the cap size. The provided section modulus for the initial cap size is:

$$\text{Provided pile cap section modulus} = S_{\text{prov}} = \frac{b_{\text{cap}} \cdot d_{\text{cap}}^2}{6}$$

$$S_{\text{prov}} = \frac{16 \cdot 16^2}{6} = 682.67 \text{ in}^3$$

#### 3. Stability Factor

Stability factor for rectangular lumber in flexure =  $C_L$

For flexural components where depth does not exceed the width of the component,  $C_L = 1.0$ .

[8.6.2]

#### 4. Adjustment Factors for Reference Design Values

Size effect factor for sawn beam lumber in flexure =  $C_F$ .

For  $d_{\text{cap}} > 12.0$  in

$$C_F = (12/d_{\text{cap}})^{1/9} = 0.97$$

[8.4.4.4]

[Eqn. 8-4.4.4-2]

[8.4.4.2]

Format conversion factor =  $C_{KF}$

$$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$$

[8.4.4.3]

Wet Service factor for Posts and Timbers sawn lumber =  $C_M$

For nominal thickness greater than 4.0 in,  $C_M = 1.0$ .

[8.4.4.9]

Time Effect Factor =  $C_\lambda$ 

[Table 8.4.4.9-1]

$$C_\lambda = 0.80$$

[Eqn. 8.4.4.1-1]

$$\begin{aligned} \text{Adjusted design value} &= F_b = F_{b0} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_\lambda \\ F_b &= 1.20 \cdot 2.94 \cdot 1.00 \cdot 0.97 \cdot 0.80 = 2.74 \text{ ksi} \end{aligned}$$

**B. Pile Cap Flexural Check**Required pile cap flexural resistance =  $M_{U(m)}$ 

For the cap to meet Strength I Limit State,  $M_{R(\text{prov})}$  must equal or exceed  $M_{U(m)}$ . As determined previously,  $M_{U(m)} = 120.39$  kip-ft

Provided pile cap factored flexural resistance:

[Eqn. 8.6.1-1]

$$\begin{aligned} M_{R(\text{prov})} &= \phi_f \cdot F_b \cdot S_{\text{prov}} \cdot C_L = 0.85 \cdot 2.74 \cdot 682.67 \cdot 1.0 \\ &= 1589.94 \text{ kip-in} = 132.49 \text{ kip-ft} \end{aligned}$$

$$M_{R(\text{prov})} = 132.49 \text{ kip-ft} \geq M_{U(m)} = 120.39 \text{ kip-ft}$$

OK

**Investigate Shear  
Resistance  
Requirements  
[8.7]**

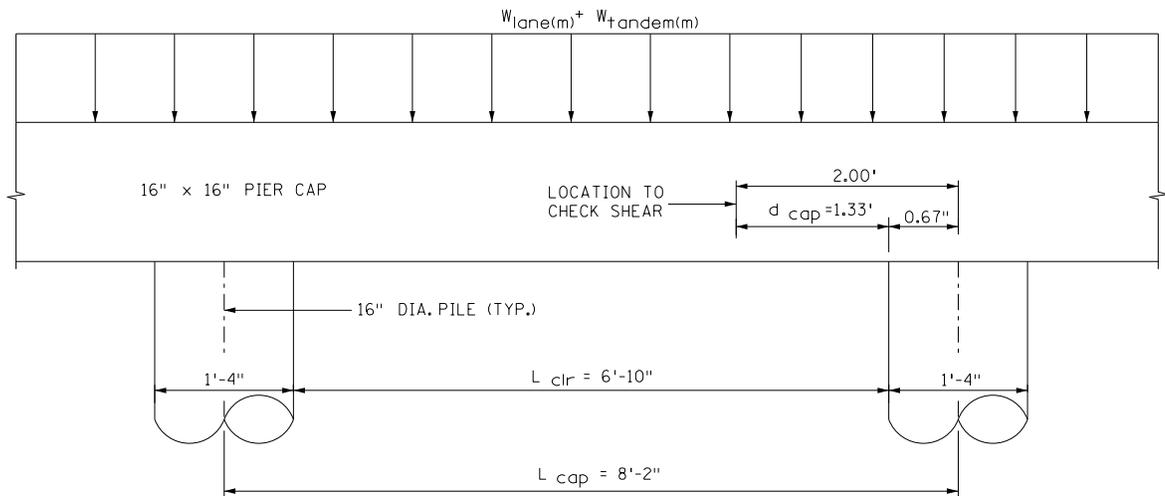
**A. Critical Shear Force Location**

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical direction, but inherent vertical resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary on this.

For components under shear, shear shall be investigated at a distance away from the face of the support equal to the depth of the component. When calculating the maximum design shear, the live load shall be placed so as to produce the maximum shear at a distance from the support equal to the lesser of either three times the depth of the component ( $d_{\text{cap}}$ ) or one-quarter of the span ( $L_{\text{cap}}$ ). This placement of the live load is more applicable when it is applied as axle point loads on longitudinal members, rather than the transverse distributed loads used in this example.

$$\begin{aligned} \text{Location to check for shear} &= (d_{\text{cap}} + \frac{1}{2} \cdot d_{\text{pile}}) / L_{\text{cap}} \\ &= (1.33 \text{ ft} + \frac{1}{2} \cdot 1.33 \text{ ft}) / 8.17 \text{ ft} \end{aligned}$$

Check for shear at about 24% of span length away from the support centerlines, or 2.00 ft



**Figure 8.7.2.3 – Cap Shear Check Location**

### B. Unfactored Shear Forces Acting on Pile Cap

These shear forces are less than the maximums listed in Table 8.7.2.1. The results given below are not the maximum shear forces on the pile cap. Rather, they are the values taken at the appropriate distance " $d_{cap}$ " from the critical support face.

#### 1. Dead Load Shear Force

Component dead load shear force at a distance " $d_{cap}$ " away from the support face =  $V_{dc} = 4.81$  kips

Wear course dead load shear force at a distance " $d_{cap}$ " away from the support face =  $V_{dw} = 4.78$  kips

#### 2. Live Load Shear Forces (Multiple Lanes Loaded)

Only the design tandem and lane loads, for the multiple lanes loaded case, are shown below. From the earlier results, this is the load case that produces the maximum shear force effect on the pier cap being analyzed.

##### a. Design Tandem Axle Loads

Design tandem shear forces at a distance " $d_{cap}$ " away from the support =  $V_{tandem(m)} = 13.81$  kips

##### b. Design Lane Load

Design lane shear force at a distance " $d_{cap}$ " from the support =  $V_{lane(m)} = 4.28$  kips

**[3.4.1]****C. Factored Shear Force Acting on Pile Cap****1. Load Modifiers**

Load modifiers for cap design are shown in the flexure check.

**2. Strength I Limit State Load Factors**

Use the Strength I Limit State to determine the required shear resistance of the pile cap.

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results (Table 8.7.2.1) indicate that multiple lanes loaded with the design tandem and lane loads control for shear.

**3. Strength I Limit State Shear Force**

Strength I Limit State factored shear force, two lanes loaded =  $V_{u(m)}$

**[Tables 3.4.1-1 and 3.4.1-2]**

$$V_{u(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot (V_{tandem(m)} + V_{lane(m)})]$$

$$V_{u(m)} = 1.0 \cdot [1.25 \cdot (4.81) + 1.50 \cdot (4.78) + 1.75 \cdot 1.0 \cdot (13.81 + 4.28)] = 44.84 \text{ kips}$$

**Check Shear Resistance of Cap****A. Factored Shear Resistance**

The factored shear force ( $V_{u(m)}$ ) is the required shear resistance of the cap that needs to be compared with the actual factored shear resistance of the cap ( $V_r$ ).

**[Eqns. 8.7-1, 8.7-2]**

For a rectangular wood section  $V_r = \phi_v \cdot F_v \cdot b_{cap} \cdot d_{cap}/1.5$

**[8.5.2.2]****1. Resistance Factor**

Shear resistance factor =  $\phi_v = 0.75$

**[8.4.4.2]****2. Adjustment Factors for Reference Design Values**

Format conversion factor:  $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$

**[8.4.4.3]**

Wet Service factor =  $C_M = 1.00$

**[8.4.4.9]**

Time effect factor =  $C_\lambda = 0.80$

**[Eqn. 8.4.4.1-2]**

Adjusted design value =  $F_v = F_{v0} \cdot C_{KF} \cdot C_M \cdot C_\lambda$

$$F_v = 0.17 \cdot 3.33 \cdot 1.00 \cdot 0.80 = 0.453 \text{ ksi}$$

**B. Pile Cap Shear Check**

Required pile cap shear resistance =  $V_{u(m)}$

For the cap to meet Strength I Limit State,  $V_{r(\text{prov})}$  must equal or exceed  $V_{u(m)}$ . As determined previously,  $V_{u(m)} = 44.84$  kips.

[Eqn. 8.7-2]

$$V_{r(\text{prov})} = \phi_v \cdot \frac{(F_v \cdot b_{\text{cap}} \cdot d_{\text{cap}})}{1.5} = 0.75 \cdot \frac{(0.453 \cdot 16 \cdot 16)}{1.5} = 57.98 \text{ kips}$$

$$V_{u(m)} = 44.84 \text{ kips} \leq V_{r(\text{prov})} = 57.98 \text{ kips} \quad \text{OK}$$

**Investigate  
Compression  
Resistance  
Requirements**

**A. Unfactored Support Reactions Acting on the Pile Cap**

The maximum support reactions are listed in Table 8.7.2.1.

**1. Dead Load Reaction Force**

Maximum component dead load reaction force =  $R_{dc} = 15.82$  kips

Maximum wear course dead load reaction force =  $R_{dw} = 15.73$  kips

**2. Live Load Reaction Forces (Multiple Lanes Loaded)**

Only the design tandem and lane load reactions, for the multiple lanes loaded case, are shown below. From the earlier results, this is the load case that produces the maximum reaction forces.

**a. Design Tandem Axle Loads**

Maximum design tandem reaction force =  $R_{\text{tandem}(m)} = 45.37$  kips

**b. Design Lane Load**

Maximum design lane reaction force =  $R_{\text{lane}(m)} = 14.06$  kips

[3.4.1]

**B. Factored Support Reaction Forces Acting on Pile Cap**

Strength I Limit State maximum factored support reaction due to two lanes loaded case =  $P_{u(m)}$

[Tables 3.4.1-1  
and 3.4.1-2]

$$P_{u(m)} = \eta \cdot [1.25 \cdot R_{dc} + 1.50 \cdot R_{dw} + 1.75 \cdot r \cdot (R_{\text{tandem}(m)} + R_{\text{lane}(m)})]$$

$$P_{u(m)} = 1.0 \cdot [1.25 \cdot (15.82) + 1.50 \cdot (15.73) + 1.75 \cdot 1.0 \cdot (45.37 + 14.06)] \\ = 147.37 \text{ kips}$$

**Check Compression  
Resistance of Cap**

**A. Factored Bearing Resistance**

The maximum factored support reaction  $P_{u(m)}$  is the required compression resistance perpendicular to the grain of the cap that needs to be compared with the actual factored compression resistance perpendicular to the grain of the cap ( $P_r$ ).

[Eqns. 8.8.1-1,  
8.8.3-1]

$$P_r = \phi_{\text{cperp}} \cdot F_{\text{cp}} \cdot A_b \cdot C_b$$

**[8.5.2.2]****1. Resistance Factor**Compression perpendicular to grain resistance factor =  $\phi_{cperp} = 0.90$ **[8.4.4.2]****2. Adjustment Factors for Reference Design Values**Format conversion factor:  $C_{KF} = 2.1/\phi = 2.1/0.90 = 2.33$ **[8.4.4.3]**Wet Service factor =  $C_M = 0.67$ **[8.4.4.9]**Time effect factor =  $C_\lambda = 0.80$ **[Eqn. 8.4.4.1-5]**Adjusted design value =  $F_{cp} = F_{cp0} \cdot C_{KF} \cdot C_M \cdot C_\lambda$  $F_{cp} = 0.625 \cdot 2.33 \cdot 0.67 \cdot 0.80 = 0.781$  ksi**3. Pile Cap Bearing Dimensions**

For this calculation contribution from other steel on the top of the pile such as the leveling ring are conservatively ignored. Only the steel pile top plate thickness of 3/8 inches is added to the pile diameter for the area considered effective for bearing resistance of the cap.

Bearing length =  $L_b = \frac{1}{2} \cdot d_{pile} = 8$  inBearing width =  $b_b = \frac{1}{2} \cdot d_{pile} = 8$  inBearing Area =  $A_b = [\pi \cdot (d_{pile})^2] / 4 = [\pi \cdot (16.75)^2] / 4 = 220.35$  in<sup>2</sup>**4. Bearing Adjustment Factor****[Table 8.8.3-1]**Adjustment Factor for Bearing =  $C_b$  $L_b = 8.0$  in  $\geq 6.0$  in  $C_b = 1.00$ **B. Pile Cap Bearing Resistance Check**Required pile cap compression resistance =  $P_{u(m)} = 147.37$  kips

For the cap to meet Strength I Limit State, provided compression resistance perpendicular to grain =  $P_{r(prov)}$  must equal or exceed  $P_{u(m)}$ .

**[Eqn. 8.8.3-1]** $P_{r(prov)} = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b = 0.9 \cdot 0.781 \cdot 220.35 \cdot 1.0 = 154.88$  kips $P_{u(m)} = 147.37$  kips  $\leq P_{r(prov)} = 154.88$  kipsOK

**8.7.3 Glulam Beam  
Superstructure  
Design Example**

This example goes through the design of glulam beams. The glulam beams are the main load carrying members for the bridge span and will have transverse timber deck panels. The last design example found in Article 8.7.4 will be for two different transverse deck types that could be used on these glulam beams to support the road surface: spike laminated deck panels, and glulam deck panels. This bridge type is also intended for use on secondary roads with low truck traffic volumes. The glulam beams being designed are intended to span from substructure to substructure.

**[8.4.1.2]**

The beams are required to be manufactured using wet use adhesives to join the individual laminates to attain the specified beam size, and under this condition the adhesive bond is stronger than the wood laminates. The beams are to be manufactured meeting the requirements of ANSI/AITC A190.1. Lamination widths for Western Species and for Southern Pine are shown in AASHTO LRFD, and the table of design values. A more complete list of beam sizes, as well as design values, is provided in the NDS.

**[Figure 8.3-1]****A. Material and Design Parameters**

The dimension annotations used throughout this design example are as follows. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications* for glulam beams ( $w_{bm}$  &  $d_{bm}$  used here). The letter notations shown in Figure 8.3-1 for sawn components will be used here for the sawn components ( $b$ ,  $d$ , etc.).

**[8.4.1.2.2]**

For glulam beams, the timber dimensions stated shall be taken as the actual net dimensions.

**1. End of Beam Support**

The ends of the glulam beams could be supported by timber pile caps or bearing pads as part of a single span or multi span bridge superstructure. For the purposes of this example, a single span superstructure supported by bearing pads on concrete substructures will be assumed. The bearing pad design is not a part of this design example, it will be assumed that the compression in the wood governs the bearing area size.

**[9.9.8]****2. Bituminous Wearing Surface**

MnDOT uses a 2% cross slope whenever practicable. In this case, a minimum thickness of 2 inches at edge of roadway (face of curb) and

6 inches thickness at centerline of the road gives an average depth of wearing course = 4 in.

However, using a constant longitudinal thickness on a bridge superstructure with glulam beams will result in a roadway surface with a hump due to the beam camber. It is preferred to construct the final top of bituminous surface uniformly in the longitudinal direction on the deck.

If the glulam beam is cambered and the top of driving surface on the bituminous is uniform, or follows the grade for a road having a straight line profile grade, the bituminous thickness must vary longitudinally. It may vary more, if for example, the profile grade has a sag vertical curve that the bituminous must accommodate. The profile grade for specific bridge designs should be reviewed to make certain the proper bituminous thickness is used in the design of the glulam beams.

For this design example, an extra 0.45 inches average bituminous thickness is assumed which is conservatively based on a straight line average. This will be verified later in this Glulam Beam Superstructure Design Example after the beam camber is calculated. Therefore, the bituminous wearing surface thickness that will be used in the dead load calculations below for the glulam beams in this design example =  $d_{ws} = 4.45$  in.

### **3. Curb and Railing** (TL-4 Glulam Timber Rail w/Curb on transv. deck)

Width of timber curb =  $b_{curb} = 12$  in

Depth of timber curb =  $d_{curb} = 6.75$  in

Width of timber rail post =  $b_{post} = 10.5$  in

Length of timber rail post =  $L_{post} = 8.75$  in

Depth of timber rail post =  $d_{post} = 37.5$  in

Width of timber spacer block =  $b_{spacer} = 3.125$  in

Length of timber spacer block =  $L_{spacer} = 8.75$  in

Depth of timber spacer block =  $d_{spacer} = 10.5$  in

Width of timber scupper =  $b_{scupper} = 12$  in

Length of timber scupper =  $L_{scupper} = 54$  in

Depth of timber scupper =  $d_{scupper} = 6.75$  in

Width of timber rail =  $b_{rail} = 8.75$  in

Depth of timber rail =  $d_{rail} = 13.5$  in

Spacing between barrier posts =  $s_{post} = 8.0$  ft = 96 in (maximum)

The timber barrier design is not a part of this design example, but the dimensions are used for weight considerations. Refer to the resources noted earlier in Article 8.5 of this manual for the TL-4 Crash Tested Bridge Rail details.

**[8.4.1.2]****4. Glulam Beams**

Assumed depth of glulam timber beams =  $d_{bm} = 46.75$  in

Assumed width of glulam timber beams =  $w_{bm} = 8.5$  in

**[8.4.1.2.2]**

Glulam beams are supplied to the dimensions specified. Attention must be given to the species of wood, as laminate sizes vary based on species.

**5. Span Lengths**

Actual longitudinal length of the beams, which is also the deck length, or bridge length =  $L = 43.50$  ft

MnDOT uses the effective span, or design span, as center to center of the beam bearing lengths. The assumed beam bearing length (18 in) is checked at the end of this Glulam Beam Superstructure Design Example.

Effective span length for the single span of glulam beams =  $L_e$

$$L_e = L - 2 \cdot \frac{1}{2} \cdot L_b = 43.50 - 2 \cdot \frac{1}{2} \cdot \frac{18}{12} = 42.0 \text{ ft}$$

**6. Unit Weights and Moisture Content****[Table 8.4.1.2.3-1]**

Type of glulam beam wood material (outer/core laminates are the same species): Southern Pine – SP/SP (24F-V3).

**[Table 3.5.1-1]**

Unit weight of soft-wood =  $\gamma_{SP} = 0.050$  kcf.

The deck will also be comprised of a soft-wood (Southern Pine or Douglas Fir). For this design example, "SP" is shown as the unit weight for the deck, but any softwood will have the same unit weight.

**[MnDOT Table 3.3.1]**

Unit weight of bituminous wearing surface =  $\gamma_{WS} = 0.150$  kcf

**[MnDOT 3.3]**

Standard MnDOT practice is to apply a future wearing course of 20 psf.

**[8.4.4.3]**

MnDOT designs for in-service wet-use only which is a MC of greater than 16% for glulam.

**[Table 8.4.1.2.3-1]****7. Southern Pine Structural Glulam (24F-V3) Strength Properties**

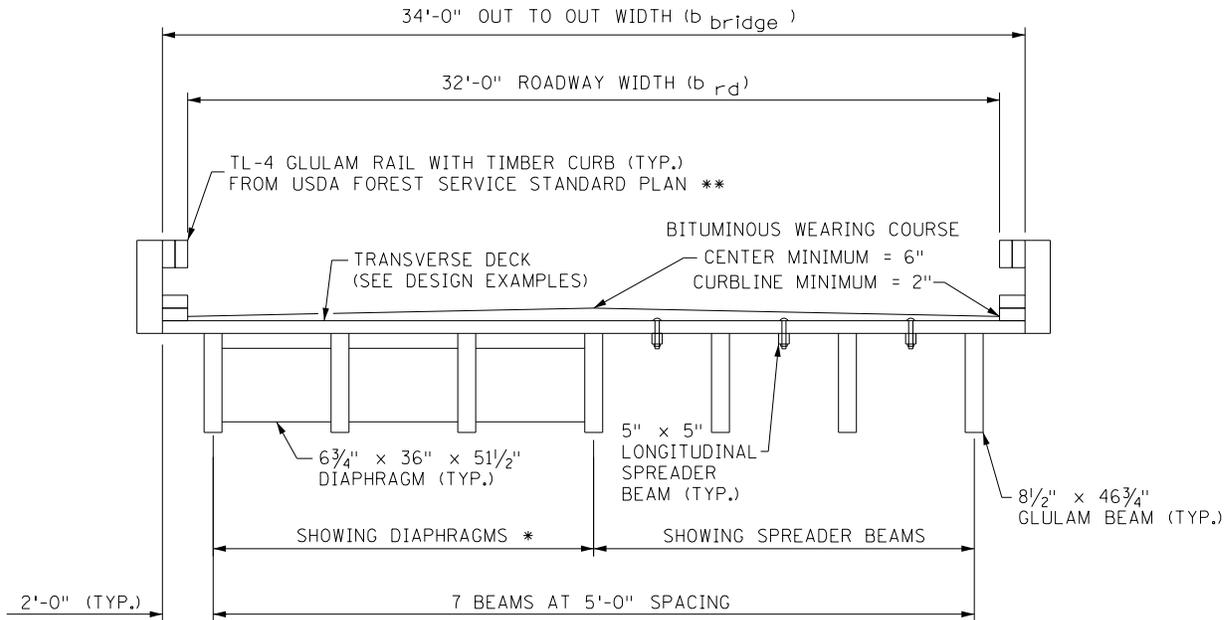
Reference Design Value for flexure =  $F_{bx0} = 2.400$  ksi

Reference Design Value for compression perpendicular to grain

=  $F_{cpo} = 0.740$  ksi (end bearing is on tension face)

Reference Design Value for shear parallel to grain =  $F_{vx0} = .300$  ksi (for checking horizontal shear)

Modulus of elasticity =  $E_{x0} = 1800$  ksi



**Figure 8.7.3.1 – Glulam Beams Layout**

\*Timber diaphragms are located near each bearing and at mid span

\*\*Rail (barrier) posts spacing is 8.0 ft

### Select the Basic Configuration

The bridge consists of 7 equally spaced glulam beams of the same size with a transverse wood deck. It is recommended to attach the deck to the beams with lag screws to stabilize the deck and prevent excess cracking in the bituminous wear course (refer to Article 8.7.4 narrative). Each glulam beam is designed as a simply supported member.

### [8.11.3]

Minimal specific guidance is provided in AASHTO LRFD for bracing requirements of glulam beams. It only states that fabricated steel shapes or solid wood blocks should be used. Wood is commonly used for blocking on wood beam bridges, and generally is less cost and easier to install than steel. Also, solid wood blocks require less design effort than designing steel and associated connectors.

For deeper glulam beams, glulam diaphragms are used to attain the appropriate depth. Traditionally transverse bracing was required to be a minimum of  $\frac{3}{4}$  the depth of a bending member and is currently specified in AASHTO LRFD for sawn wood beams, so that can be used as a guide on current glulam beam designs. The maximum spacing of 25.0 ft for sawn beams can also be used as a guide for standard glulam beam designs. The designer needs to check that lateral stability requirements for bending members are being met for individual designs.

### [9.9.4.3]

**B. Panel Dimensions and Bridge Width Deck**

The transverse deck design example is found in Article 8.7.4 of this manual. It includes both a design for a spike laminated deck panel assembled from sawn lumber and a design for a deck panel that is glulam. For glulam the dimensions are taken as the actual net dimensions. The sawn lumber is typically surfaced one side and one edge, and so the nominal deck thickness dimension is used for dead load.

The spike laminated deck thickness of 6 inches is used for the deck dead load in this glulam beam design example because that has a larger dead load effect than the glulam deck. The spike laminated deck also causes the live load fraction on the beam to be larger than with a glulam deck, and so creates the worst case force effects of the two deck types for the beam design.

The transverse deck design example incorporates the use of a longitudinal stiffener beam, or spreader beam, for the deck panels to be considered interconnected in accordance with AASHTO LRFD. The dead load of the spreader beam will be included in the deck dead load for this glulam beam design example, and the size determination (5 in x 5 in) for the spreader beam is shown in the transverse deck design example.

Length of bridge deck panels =  $b_1 = 34.0$  ft

Overall width of bridge deck =  $b_{\text{bridge}} = 34.0$  ft

Width of each timber barrier =  $b_{\text{barrier}} = 1.0$  ft

Width of roadway =  $b_{\text{rd}} = b_{\text{bridge}} - 2 \cdot b_{\text{barrier}} = 34.0 - (2 \cdot 1) = 32.0$  ft

**C. Beam Spacing Dimensions**

The exterior beam should generally be near enough to the outside deck edge so that the deck overhang and the exterior beam do not govern the respective designs. However, economy is gained by not placing the beam at the outside deck edge (possibly less total beams required).

**[3.6.1.3]**

Looking at AASHTO LRFD for the application of vehicular live load, the tire on a truck axle is basically placed 1.0 ft from the face of curb or railing for deck design, and 2.0 ft for the design of all other components. Using the 1.0 ft for deck design, the tire would occur 2.0 ft from the edge deck, and so if a beam is placed here the outside deck cantilever will not govern. Typically the exterior beam then would also not govern, because applying the 2.0 ft for the design of all other components the tire on the axle would occur inside of the exterior beam. For this design example, a

2.0 ft overhang each side measured from center of the exterior beam to edge of deck will be tried.

**[Table 4.6.2.2a-1]**

The live load distribution to an interior beam is determined from the table in AASHTO LRFD. The range of applicability for this table is a maximum beam spacing of 6.0 ft. A beam spacing of 5.0 ft fits within this range, and so that will be tried for this glulam beam design example.

**Determine Dead and Live Load Bending Moments**

**A. Dead Loads per Beam**

The units for the dead load results are given in kips per foot for one beam. MnDOT assumes that the barrier load for all wood structure types acts uniformly over the bridge width. Deck and wear course are calculated based on tributary area for simplicity, as the exterior beam generally will not govern for typical designs. Exterior beam loads are shown in the design example to illustrate that the exterior beam will not govern the design.

**1. Dead Loads per longitudinal foot**

$$\begin{aligned} \text{Weight of beam} = W_{\text{beam}} &= \gamma_{\text{SP}} \cdot d_{\text{bm}} \cdot W_{\text{bm}} = 0.050 \cdot 46.75/12 \cdot 8.5/12 \\ &= 0.138 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Weight of deck, interior beams (including spreader beam)} \\ = W_{\text{deck\_int}} &= \gamma_{\text{SP}} \cdot d_{\text{deck}} \cdot S_{\text{int\_bm}} + \gamma_{\text{SP}} \cdot d_{\text{spdr}} \cdot b_{\text{spdr}} \\ &= (0.050 \cdot 6/12 \cdot 5.0) + (0.050 \cdot 5/12 \cdot 5/12) = 0.134 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Weight of deck, exterior beams (including spreader beam)} \\ = W_{\text{deck\_ext}} &= \gamma_{\text{SP}} \cdot d_{\text{deck}} \cdot S_{\text{ext\_bm}} + \gamma_{\text{SP}} \cdot d_{\text{spdr}} \cdot b_{\text{spdr}} \cdot 1/2 \\ &= (0.050 \cdot 6/12 \cdot 4.5) + (0.050 \cdot 5/12 \cdot 5/12 \cdot 1/2) = 0.117 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Weight of wearing surface, interior beams} = W_{\text{ws\_int}} &= \gamma_{\text{ws}} \cdot d_{\text{ws}} \cdot S_{\text{int\_bm}} \\ &= 0.150 \cdot 4.45/12 \cdot 5.0 = 0.278 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Weight of wearing surface, exterior beams} = W_{\text{ws\_ext}} &= \gamma_{\text{ws}} \cdot d_{\text{ws}} \cdot S_{\text{ext\_bm}} \\ &= 0.150 \cdot 3.0/12 \cdot 3.5 = 0.131 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Weight of future wearing course, interior beams} = W_{\text{FWC}} \cdot S_{\text{int\_bm}} \\ = 0.020 \cdot 5 = 0.100 \text{ klf} \end{aligned}$$

$$\begin{aligned} \text{Weight of future wearing course, exterior beams} = W_{\text{FWC}} \cdot S_{\text{ext\_bm}} \\ = 0.020 \cdot 3.5 = 0.070 \text{ klf} \end{aligned}$$

**2. Determine linear weight of rail system elements.**

Volume of timber curb per foot of bridge length =  $V_{\text{curb}}$

$$V_{\text{curb}} = (b_{\text{curb}} \cdot d_{\text{curb}} \cdot 12 \text{ in/ft}) = (12 \cdot 6.75 \cdot 12) = 972.0 \text{ in}^3/\text{ft}$$

Volume of rail post and spacer block per foot of bridge length =  $V_{\text{post}}$

$$V_{\text{post}} = (b_{\text{post}} \cdot L_{\text{post}} \cdot d_{\text{post}} + b_{\text{spacer}} \cdot L_{\text{spacer}} \cdot d_{\text{spacer}}) / S_{\text{post}}$$

$$V_{\text{post}} = [(10.5 \cdot 8.75 \cdot 38) + (3.125 \cdot 8.75 \cdot 10.5)] / 8 \\ = 472.3 \text{ in}^3/\text{ft}$$

Volume of scupper per foot of bridge length =  $V_{\text{scupper}}$

$$V_{\text{scupper}} = (b_{\text{scupper}} \cdot L_{\text{scupper}} \cdot d_{\text{scupper}}) / S_{\text{post}}$$

$$V_{\text{scupper}} = (12 \cdot 54 \cdot 6.75) / 8 = 546.75 \text{ in}^3/\text{ft}$$

Volume of timber rail per foot of bridge length =  $V_{\text{rail}}$

$$V_{\text{rail}} = (b_{\text{rail}} \cdot d_{\text{rail}} \cdot 12 \text{ in/ft}) = (8.75 \cdot 13.5 \cdot 12) = 1417.5 \text{ in}^3/\text{ft}$$

Volume of timber railing per longitudinal foot of bridge length =  $V_{\text{barrier}}$

$$V_{\text{barrier}} = V_{\text{curb}} + V_{\text{post}} + V_{\text{scupper}} + V_{\text{rail}}$$

$$V_{\text{barrier}} = 972.0 + 472.3 + 546.75 + 1417.5 = 3408.6 \text{ in}^3/\text{ft} \\ = 1.973 \text{ ft}^3/\text{ft}$$

Total linear weight of combined timber curbs/railings =  $W_{\text{barrier}}$

$$W_{\text{barrier}} = \frac{2 \cdot \gamma_{\text{DFL}} \cdot V_{\text{barrier}}}{\text{beams}_{\text{total}}} = \frac{2 \cdot 0.050 \cdot 1.973}{7} = 0.028 \text{ klf}$$

This linear weight result assumes that the curb/railing weight acts uniformly over the entire deck width.

### 3. Diaphragm point loads

$$\text{Volume of diaphragm} = V_{\text{diaph}} = b_{\text{diaph}} \cdot L_{\text{diaph}} \cdot d_{\text{diaph}} \\ = (51.5 \cdot 6.75 \cdot 36) / 1728 = 7.242 \text{ ft}^3$$

$$\text{Diaphragm load, interior beams} = P_{\text{diaph\_int}} = \gamma_{\text{DFL}} \cdot V_{\text{diaph}} \\ = 0.050 \cdot 7.242 = 0.362 \text{ kips}$$

$$\text{Diaphragm load, exterior beams} = P_{\text{diaph\_ext}} = (\gamma_{\text{DFL}} \cdot V_{\text{diaph}}) / 2 \\ = (0.050 \cdot 7.242) / 2 = 0.181 \text{ kips}$$

## B. Dead Load Bending Moments per Beam

### 1. Moments of Individual loads

[AISC 14<sup>th</sup> p. 3-213]

Maximum bending moment due to beam weight

$$M_{\text{beam}} = \frac{w_{\text{bm}} \cdot (L_e)^2}{8} = \frac{0.138 \cdot 42.0^2}{8} = 30.43 \text{ kip-ft}$$

Maximum bending moment due to deck weight, interior beams

$$M_{\text{deck\_int}} = \frac{w_{\text{deck\_int}} \cdot (L_e)^2}{8} = \frac{0.134 \cdot 42.0^2}{8} = 29.55 \text{ kip-ft}$$

$$w_{\text{deck\_ext}} \cdot (L_e)^2 = 0.058 \cdot 21.33^2 \quad \text{kip-ft}$$

Maximum bending moment due to deck weight, exterior beams

$$M_{\text{deck\_ext}} = \frac{w_{\text{deck\_ext}} \cdot (L_e)^2}{8} = \frac{0.117 \cdot 42.0^2}{8} = 25.80 \text{ kip-ft}$$

Maximum bending moment due to wearing surface, interior beams

$$M_{\text{ws\_int}} = \frac{w_{\text{ws\_int}} \cdot (L_e)^2}{8} = \frac{0.278 \cdot 42.0^2}{8} = 61.30 \text{ kip-ft}$$

Maximum bending moment due to wearing surface, exterior beams

$$M_{\text{ws\_ext}} = \frac{w_{\text{ws\_ext}} \cdot (L_e)^2}{8} = \frac{0.131 \cdot 42.0^2}{8} = 28.89 \text{ kip-ft}$$

Maximum bending moment due to future wearing course, interior beams

$$M_{\text{FWC\_int}} = \frac{w_{\text{FWC\_int}} \cdot (L_e)^2}{8} = \frac{0.100 \cdot 42.0^2}{8} = 22.05 \text{ kip-ft}$$

Maximum bending moment due to future wearing course, exterior beams

$$M_{\text{FWC\_ext}} = \frac{w_{\text{FWC\_ext}} \cdot (L_e)^2}{8} = \frac{0.070 \cdot 42.0^2}{8} = 15.44 \text{ kip-ft}$$

Maximum bending moment due to diaphragm weight, interior beams

$$M_{\text{diaph\_int}} = \frac{P_{\text{diaph\_int}} \cdot L_e}{4} = \frac{0.362 \cdot 42.0}{4} = 3.80 \text{ kip-ft}$$

Maximum bending moment due to diaphragm weight, exterior beams

$$M_{\text{diaph\_ext}} = \frac{P_{\text{diaph\_ext}} \cdot L_e}{4} = \frac{0.181 \cdot 42.0}{4} = 1.90 \text{ kip-ft}$$

Maximum bending moment due to curb/railing weight =  $M_{\text{barrier}}$

$$M_{\text{barrier}} = \frac{w_{\text{barrier}} \cdot (L_e)^2}{8} = \frac{0.028 \cdot 42.0^2}{8} = 6.17 \text{ kip-ft}$$

## 2. Sum of Dead Load Moments per Beam

### a. Interior Beam

Maximum bending moment due to bridge component dead loads, interior beam

$$\begin{aligned} M_{\text{dc\_int}} &= M_{\text{beam}} + M_{\text{deck\_int}} + M_{\text{diaph\_int}} + M_{\text{barrier}} \\ &= 30.43 + 29.55 + 3.80 + 6.17 = 69.95 \text{ kip-ft} \end{aligned}$$

Maximum bending moments due to wearing surface loads, interior beam

$$\begin{aligned} M_{\text{dw\_int}} &= M_{\text{ws\_int}} + M_{\text{FWC}} \\ &= 61.30 + 22.05 = 83.35 \text{ kip-ft} \end{aligned}$$

**b. Exterior Beam**

Maximum bending moment due to bridge component dead loads, exterior beam

$$\begin{aligned} M_{dc\_ext} &= M_{beam} + M_{deck\_ext} + M_{diaph\_ext} + M_{barrier} \\ &= 30.43 + 25.80 + 1.90 + 6.17 = 64.30 \text{ kip}\cdot\text{ft} \end{aligned}$$

Maximum bending moments due to wearing surface loads, exterior beam

$$\begin{aligned} M_{dw\_ext} &= M_{ws\_ext} + M_{FWC} \\ &= 28.89 + 15.44 = 44.33 \text{ kip}\cdot\text{ft} \end{aligned}$$

**[3.6.1.2]****C. Live Load Bending Moments**

The live load bending moment will be calculated per lane (12 ft) and later converted to a per beam format.

**[3.6.1.2.2]****1. Design Truck Axle Loads**

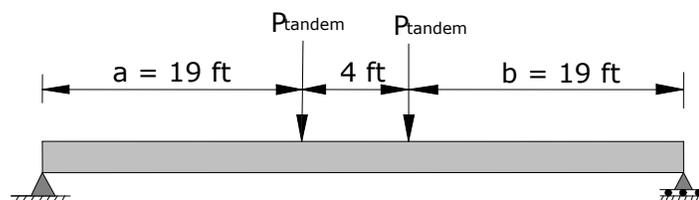
Point loads and spacing of the design truck axles are shown in AASHTO LRFD Figure 3.6.1.2.2-1.

Maximum bending moment due to design truck axle load =  $M_{truck}$ . This truck moment is available in multiple reference tables (including Table 3.4.1.2 in this manual) for a 42.0 ft span.

$$M_{truck} = 485.2 \text{ kip}\cdot\text{ft}$$

**[3.6.1.2.3]****2. Design Tandem Axle Loads**

Point load of design tandem axle =  $P_{tandem} = 25$  kips, spaced at 4 ft.



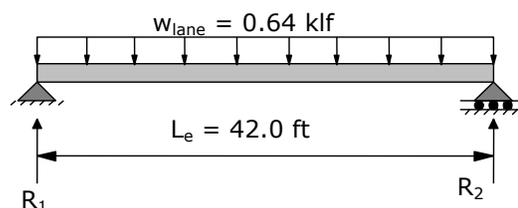
Maximum bending moment due to design tandem axle loads =  $M_{tandem}$

**[AISC 14<sup>th</sup> p. 3-228]**

$$M_{tandem} = Pa = 25.0 \cdot 19.0 = 475.0 \text{ kip}\cdot\text{ft}$$

**[3.6.1.2.4]****3. Design Lane Loads**

Uniform design lane load =  $w_{lane} = 0.64$  klf



Maximum bending moment due to design lane load =  $M_{\text{lane}}$

$$M_{\text{lane}} = \frac{w_{\text{lane}} \cdot (L_e)^2}{8} = \frac{0.64 \cdot 42^2}{8} = 141.1 \text{ kip-ft}$$

**[4.6.2.2]**

**D. Live Load Distribution**

The live load bending moments, calculated above, need to be distributed to a per beam basis.

The transverse deck design example next in the Chapter after this beam design example includes both a design for a spike laminated deck panel assembled from sawn lumber and a design for a deck panel that is glulam. A spike laminated deck gives a higher wheel load fraction and so that will be used for this beam design example (it is the worst case).

**[3.6.1.1.1]**

Maximum number of traffic lanes on the deck =  $N_L$

$$N_L = \frac{b_{\text{rd}}}{12 \frac{\text{ft}}{\text{lane}}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

**[Table 4.6.2.2a-1]**

Live Load Distribution Factor ( $g_{\text{int}}$ ) for interior beams is calculated using beam spacing ( $S$ ), and is based on deck type and number of loaded lanes.

**[3.6.1.1.2]**

The multiple presence factors are not intended to be applied in conjunction with the load distribution factors specified in Table 4.6.2.2a-1. The multiple presence factors have been accounted for in these equations.

**[Table 4.6.2.2a-1]**

Two or more design lanes loaded is compared with one design lane loaded to determine the Live Load Distribution Factor to use here.

Two or more design lanes loaded:

$$g_{\text{int}} = \frac{S}{8.5} = 0.59 \text{ Design Truck, interior beam}$$

One design lane loaded:

$$g_{\text{int}} = \frac{S}{8.3} = 0.60 \text{ Design Truck, interior beam}$$

One lane loaded gives the higher live load distribution to an interior beam, and so the interior Live Load Distribution Factor =  $g_{\text{int}} = 0.60$ .

**[4.6.2.2d]**

Typically the live load flexural moment for exterior beams is determined by applying the Live Load Distribution Factor (LLDF) specified for exterior beams. For this design example, the specified exterior Live Load Distribution Factor,  $LLDF_{\text{ext}}$ , is the lever rule.

**[3.6.1.3]**

The design vehicle is to be placed no closer than 2.0 ft from the edge of the design lane. The most severe force effect is with the edge of design lane at the face of the timber curb. For this design example, this would place one tire (0.50 Design Trucks) 1.0 ft inside of the beam and the other inside of the next beam (which is then ignored for the lever rule applied to the exterior beam).

**[C3.6.1.1.2]**

When using the lever rule, the multiple presence factor must be applied manually.

**[Table 3.6.1.1.2-1]**

Similar as for the Live Load Distribution Factor for the interior beams, one lane loaded produces the largest force effect on the exterior beams, with the multiple presence factor  $m = 1.20$  applied to the  $LLDF_{ext}$ .

**[Table 4.6.2.2d-1]**

Exterior Live Load Distribution Factor =  $g_{ext} = LLDF_{ext} \times m$ .

$$g_{ext} = \frac{0.50 \text{ Design Truck} \cdot 4\text{ft}}{5\text{ft}} \cdot 1.20 = 0.48 \text{ Design Truck, exterior beam}$$

It can be seen that as originally assumed above in "Select the Basic Configuration", the interior beam will have the more severe live load force effect.

**E. Live Load Moments per Beam****a. Interior Beam**

Maximum moments from design truck load single lane =  $M_{truck(s)}$

$$M_{truck(s)} = M_{truck} \cdot g_{int} = 485.2 \cdot 0.60 = 291.12 \text{ kip-ft}$$

Maximum moment from design tandem load single lane =  $M_{tandem(s)}$

$$M_{tandem(s)} = M_{tandem} \cdot g_{int} = 475.0 \cdot 0.60 = 285.00 \text{ kip-ft}$$

Maximum moment from design lane load single lane =  $M_{lane(s)}$

$$M_{lane(s)} = M_{lane} \cdot g_{int} = 141.1 \cdot 0.60 = 84.66 \text{ kip-ft}$$

**b. Exterior Beam**

Because  $g_{ext} < g_{int}$  as checked above in Part D., exterior beam live load moments will not be calculated.

## F. Summary of Unfactored Dead and Live Load Bending Moments per Beam

**Table 8.7.3.1 - Applied Bending Moments**

Unfactored Load Case	Maximum Positive Bending Moment (kip-ft)
Dead Loads (interior beam)	
Bridge Components ( $M_{dc}$ )	69.95
Bridge Wearing Surface ( $M_{dw}$ )	83.35
Dead Loads (exterior beam)	
Bridge Components ( $M_{dc}$ )	64.30
Bridge Wearing Surface ( $M_{dw}$ )	44.33
Live Loads (interior beam, for single lane)	
Design Truck	291.12
Design Tandem	285.00
Design Lane	84.66

## G. Factored Bending Moment per Beam

### 1. Load Modifiers

Standard MnDOT Load Modifiers are summarized in Table 3.2.1 of this manual.

For timber bridges  $\eta_D = 1.0$ . MnDOT considers four or more beams to have a conventional level of redundancy and uses  $\eta_R = 1.0$ . This example bridge is assumed to have a design ADT of over 500 for  $\eta_T = 1.0$ .

**[1.3.2]**

Therefore, importance, redundancy, and ductility factors =  $\eta = 1.0$

### 2. Strength I Limit State Load Factors

**[3.4.1]**

Use the Strength I Limit State to determine the required resistance for the beams.

**[3.6.2.3]**

Impact factor need not be applied to wood components.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis showed that the design truck load controls the bending moment of the beams. Additionally, the analysis determined that the interior beams will govern with one lane loaded. Therefore, use the design truck load with the uniform lane load in determining the critical live load bending moment acting on the interior beams.

Also, the earlier analysis calculated dead load bending moment on both the interior and exterior beams. The bending moments from dead load are larger on the interior beams. Strength checks only need to be done for the interior beams, since all beams shall be the same size.

[4.6.2.2.1]

### 3. Strength I Limit State Bending Moment per Beam

Factored bending moment for two lanes loaded case =  $M_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot [M_{truck(m)} + M_{lane(m)}]]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot 69.95 + 1.50 \cdot 83.35 + 1.75 \cdot 1.0 \cdot [291.12 + 84.66]] = 870.08 \text{ kip-ft}$$

**Check Flexural Resistance of Beams**

#### A. Factored Flexural Resistance

The factored bending moment ( $M_{u(m)}$ ) is the required flexural resistance of the beam that needs to be compared with the actual factored flexural resistance of the beam ( $M_r$ ).

[8.6.2]

For a rectangular wood section  $M_r = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$ .

[8.5.2.2]

#### 1. Resistance Factors

Flexural resistance factor =  $\phi_f = 0.85$

Compression perpendicular to grain resistance factor =  $\phi_{cperp} = 0.90$

#### 2. Provided Section Modulus

The section modulus is dependent on the beam size. The provided beam section modulus is determined from the beam dimensions assumed at the start of the design example.

The provided beam section modulus =  $S_{prov} = \frac{W_{bm} \cdot d_{bm}^2}{6}$

$$S_{prov} = \frac{8.5 \cdot 46.75^2}{6} = 3096.21 \text{ in}^3$$

[8.6.2]

#### 3. Stability Factor

Stability factor for the glulam beams in flexure =  $C_L$ . The stability factor shall not be applied simultaneously with the volume factor for structural glued laminated timber. In this case the beams are laterally supported and so the Stability Factor  $C_L = 1.0$ . The volume factor will be the lesser of the two values and is what will be used in the adjusted design value.

#### 4. Adjustment Factors for Reference Design Value

[8.4.4.2]

Format conversion factor for component in flexure =  $C_{KF}$

$$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$$

[8.4.4.3]

Wet Service factor for glued laminated timber in flexure =  $C_M$

[Table 8.4.4.3-2]

For structural glulam, wet service condition  $C_M = 0.80$

**[8.4.4.5]**

Volume factor for structural glulam timber in flexure, when loads are applied to wide face of laminations =  $C_V$  ( $a = 0.05$  for Southern Pine). The beams for this design example are not tension reinforced which represent the most commonly used beam type in Minnesota.

**[Eqn. 8.4.4.5-1]**

$$C_V = \left[ \left( \frac{12}{d_{bm}} \right) \cdot \left( \frac{5.125}{W_{bm}} \right) \cdot \left( \frac{21}{L_e} \right) \right]^a \leq 1.0$$

$$C_V = \left[ \left( \frac{12}{46.75} \right) \cdot \left( \frac{5.125}{8.5} \right) \cdot \left( \frac{21}{42} \right) \right]^{0.05} = 0.88$$

**[8.4.4.9]****[Table 8.4.4.9-1]**

Time effect factor for Strength I Limit State =  $C_\lambda$

$$C_\lambda = 0.80$$

**[Eqn. 8.4.4.1-1]**

Adjusted design value =  $F_b = F_{bx0} \cdot C_{KF} \cdot C_M \cdot C_V \cdot C_\lambda$

$$F_b = 2.400 \cdot 2.94 \cdot 0.80 \cdot 0.88 \cdot 0.80 = 3.97 \text{ ksi}$$

**B. Beam Flexural Check**

Required beam flexural resistance =  $M_{u(m)}$

For the beam to meet Strength I Limit State,  $M_R$  must equal or exceed  $M_{u(m)}$ . As determined previously,  $M_{u(m)} = 870.08 \text{ kip}\cdot\text{ft}$

Provided beam factored flexural resistance:

$$\begin{aligned} M_{R(\text{prov})} &= \phi_f \cdot F_b \cdot S_{\text{prov}} \cdot C_L = 0.85 \cdot 3.97 \cdot 3096.21 \cdot 1.0 \\ &= 10,448.16 \text{ kip}\cdot\text{in} = 870.68 \text{ kip}\cdot\text{ft} \end{aligned}$$

$$M_{u(m)} = 870.08 \text{ kip}\cdot\text{ft} \leq M_{R(\text{prov})} = 870.68 \text{ kip}\cdot\text{ft}$$

OK

The required beam size indicates that the originally assumed beam size can be used, based on calculations using the worst case effect of the two deck types. Next, the beam size will be checked against deflection limits.

**Investigate****Deflection****Requirements****[8.5.1]****[3.6.1.3.2]****[2.5.2.6.2]****A. Beam Live Load Deflection with Current Parameters**

The midspan deflections are to be taken as the larger of the design truck or 25% of the design truck applied in conjunction with the design lane load.

Deflections are to be calculated using Service I Limit State.

With all design lanes loaded, it is allowed to assume all supporting components deflect equally for straight girder systems.

Then, the deflection distribution factor,  $DF_{\Delta}$ , is determined as follows.

$$DF_{\Delta} = m \cdot \frac{(\# \text{ of lanes})}{(\# \text{ of beamlines})}$$

[Table 3.6.1.1.2-1]

$$\text{for } m = 1.0 \text{ (2 lanes loaded), } DF_{\Delta} = 1.0 \cdot \frac{2}{7} = 0.286$$

[2.5.2.6.2]

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

[C2.5.2.6.2]

### 1. Beam Stiffness

Moment of inertia of one beam =  $I_{\text{prov}}$

$$I_{\text{prov}} = \frac{1}{12} \cdot W_{\text{bm}} \cdot d_{\text{bm}}^3 = \frac{1}{12} \cdot 8.5 \cdot (46.75)^3 = 72,374 \text{ in}^4$$

[Table 8.4.4.3-2]

Beam modulus of elasticity with wet service included =  $E$ , ( $C_M = 0.833$ )

[Eqn. 8.4.4.1-6]

$$E = E_o \cdot C_M = 1800 \text{ ksi} \cdot 0.833 = 1499.4 \text{ ksi}$$

### 2. Live Loads

The truck deflection can be calculated with a beam program, or alternatively there are various tables available. One method is the use of a coefficient that is divided by  $EI_{\text{prov}}$ .

Design truck load used for deflection calculations =  $P_{\Delta\text{truck}}$

$$\text{Coefficient for a 42.0 ft span} = P_{\Delta\text{truck}} = 1.468 \times 10^{11}$$

(from reference 3 in Article 8.6 of this manual)

Design lane load used for deflection calculations =  $W_{\Delta\text{lane}}$

$$W_{\Delta\text{lane}} = 0.64 \text{ klf}$$

### 3. Live Load Deflection Calculations

Deflection at beam midspan due to the design truck load =  $\Delta_{\text{truck}}$

[3.6.1.3.2]

[AISC 14<sup>th</sup> p. 3-213]

$$\Delta_{\text{truck}} = DF_{\Delta} \cdot \frac{P_{\Delta\text{truck}}}{E \cdot I_{\text{prov}}} = 0.286 \cdot \frac{1.468 \times 10^{11}}{1499.4 \cdot 72,374} = 0.387 \text{ in}$$

Deflection at beam midspan due to the design lane load =  $\Delta_{\text{lane}}$

$$\Delta_{\text{lane}} = DF_{\Delta} \cdot \frac{5 \cdot W_{\Delta\text{lane}} \cdot L_e^4}{384 \cdot E \cdot I_{\text{prov}}} = 0.286 \cdot \frac{5 \cdot \frac{0.64}{12} \cdot (42.0 \cdot 12)^4}{384 \cdot 1499.4 \cdot 72,374} = 0.118 \text{ in}$$

Deflection at beam midspan due to a combination of truck (25%) and design lane load =  $\Delta_{\text{combined}}$

$$\Delta_{\text{combined}} = (0.25 \cdot \Delta_{\text{truck}}) + \Delta_{\text{lane}} = (0.25 \cdot 0.387) + 0.118$$

$$\Delta_{\text{combined}} = 0.215 \text{ in} \leq \Delta_{\text{truck}} = 0.387 \text{ in}$$

Therefore, the maximum deflection between the combination load deflection and the truck load deflection =  $\Delta = \Delta_{\text{truck}} = 0.387 \text{ in}$

### [2.5.2.6.2]

Live load deflection limit at beam midspan =  $\Delta_{\text{max}}$

$$\Delta_{\text{max}} = L_e / 425 = 42.0 / 425 = 0.0988 \text{ ft} = 1.186 \text{ in}$$

$$\Delta = 0.387 \text{ in} \leq \Delta_{\text{max}} = 1.186 \text{ in}$$

OK

The initial beam size and grade are adequate for deflection.

### Determine Camber Requirements

#### A. Beam Camber

Glulam beams are cambered because the spans are relatively long (compared to a longitudinal deck bridge). The dimension of the dead load deflection is larger and can present a look that the bridge is overloaded and sagging, and so camber counteracts the dead load deflection and the visual appearance of the deflection. The camber must also account for longer term deflection because wood is susceptible to creep. Glulam beams can be cambered in the shop without much difficulty.

### [8.12.1]

Glued Laminated timber girders shall be cambered a minimum of two times the dead load deflection at the Service Limit State.

The deflection from the total unfactored dead load is calculated. The camber will be calculated for the interior beams, and the same camber applied to the exterior beams. FWC is included here. Some judgment can be used by the designer, but for aesthetic reasons, generally slight additional extra camber is preferred over not enough camber.

Uniform distributed Dead Load:

$$W_{\Delta} = W_{\text{beam}} + W_{\text{deck\_int}} + W_{\text{ws\_int}} + W_{\text{FWC\_int}} + W_{\text{barrier}}$$

$$W_{\Delta} = 0.138 + 0.134 + 0.278 + 0.100 + 0.028 = 0.678 \text{ kip/ft}$$

Point Dead Load: (diaphragm load):  $P_{\Delta} = P_{\text{dc\_int}} = 0.362 \text{ kip}$

$$\Delta_{\text{DL}} = \frac{5 \cdot w_{\Delta} \cdot L^4}{384 \cdot E \cdot I_{\text{prov}}} + \frac{P_{\Delta} \cdot L^3}{48 \cdot E \cdot I_{\text{prov}}}$$

$$\Delta_{\text{DL}} = \frac{5 \cdot (0.678/12) \cdot (42.0 \times 12)^4}{384 \cdot 1499.4 \cdot 72,374} + \frac{0.362 \cdot (42.0 \times 12)^3}{48 \cdot 1499.4 \cdot 72,374} = 0.446 \text{ in}$$

$$\text{Camber} = 2\Delta_{DL} = 2 \cdot 0.446 = 0.89 \text{ in}$$

The initial assumption of an additional 0.45 inches of average bituminous thickness assumed early in the example, to accommodate the beam camber, is acceptable.

**Investigate Shear  
Resistance  
Requirements  
[8.7]**

**A. Critical Shear Force Location**

For components under shear, shear shall be investigated at a distance away from the face of the support equal to the depth of the component.

When calculating the maximum design shear, the live load shall be placed so as to produce the maximum shear at a distance from the support equal to the lesser of either three times the depth of the component ( $d_{\text{beam}}$ ) or one-quarter of the span ( $L_{\text{beam}}$ ).

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical direction, but inherent vertical resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary on this.

Bearing has not yet been checked, but the shear calculation typically is not critical for a larger glulam beam. For the location to check shear, it will conservatively be assumed the total bearing length is 12 in.

$$\begin{aligned} \text{Location to check for shear} &= [d_{\text{beam}} + \frac{1}{2} \cdot L_{\text{bearing}}] / L_{\text{beam}} \\ &= [3.90 \text{ ft} + \frac{1}{2} \cdot 1.0 \text{ ft}] / 42.0 \text{ ft} = 0.10 \end{aligned}$$

Check for shear at 10% of the span length away from the support centerlines.

**B. Unfactored Shear Forces Acting on the Beam**

Dead loads and live loads are positioned at different locations for calculating shear forces in a longitudinal beam.

**1. Dead Load Shear Force per Interior Beam**

The maximum shear force at the support will be calculated first. As previously shown, the interior beam is the worst case for dead load and so the exterior will not be checked.

$$V_{dc\_max} = V_{\text{beam}} + V_{\text{deck\_int}} + V_{\text{diaph\_int}} + V_{\text{barrier}}$$

$$V_{dc\_max} = 2.90 + 2.81 + 0.18 + 0.59 = 6.48 \text{ kips}$$

$$V_{dw\_max} = V_{ws} + V_{FWC}$$

$$V_{dw\_max} = 5.84 + 2.10 = 7.94 \text{ kips}$$

Component dead load shear force at a distance " $d_{beam}$ " away from the support face =  $V_{dc} = 0.80 \cdot 6.48 = 5.18 \text{ kips}$

Wear course dead load shear force at a distance " $d_{beam}$ " away from the support face =  $V_{dw} = 0.80 \cdot 7.94 = 6.35 \text{ kips}$

## 2. Live Load Shear Force per Interior Beam

[Eqn. 4.6.2.2.2a-1]

The live load shear is distributed based on an average of: (0.60 of an undistributed wheel load) added to (the distribution specified in Table 4.6.2.2.2a-1). The live load is positioned as specified above.

Check position on beam: lesser of  $3 \cdot d_{beam}$  or  $L_e / 4$

$$3 \cdot d_{beam} = 3 \cdot 3.90 = 11.70 \text{ ft}$$

$$L_e / 4 = 42.0 / 4 = 10.50 \text{ ft}$$

Use 10.50 ft from the centerline of bearing to position the live load.

### a. Design Tandem Axle Loads

Design tandem shear forces with the live load placed at a distance away from the support of 10.50 ft =  $V_{tandem}$

$$V_{tandem} = \frac{25 \cdot (31.5 + 27.5)}{42.0} = 35.12 \text{ kips}$$

$$V_{tandem} = 35.12 \text{ kips}$$

### b. Design Truck Axle Loads

Design truck shear forces with the live load placed at a distance away from the support of 10.50 ft =  $V_{truck}$

$$V_{truck} = \frac{32 \cdot (31.5 + 17.5)}{42.0} + \frac{8 \cdot (3.5)}{42.0} = 38.00 \text{ kips}$$

$$V_{truck} = 38.00 \text{ kip (controls for live load)}$$

### c. Design Lane Load

Design lane load shear forces at a distance away from the support of 10.50 ft =  $V_{lane}$

$$V_{lane} = 0.50 \times 13.44 = 6.72 \text{ kips}$$

**d. Live Load per Interior Beam**

$$V_{LL} = 0.50[(0.60 V_{LU}) + V_{LD}]; \text{ use } g_{int} = 0.60 \text{ from Table 4.6.2.2.2a-1}$$

Shear live loads are multiplied by 0.50 for undistributed wheel loads,  $V_{LU}$

$$V_{LL} = 0.50[(0.60 \cdot 0.50(38.00 + 6.72) + (38.00 + 6.72)0.60]$$

$$V_{LL} = 20.12 \text{ kips}$$

**[3.4.1]****C. Factored Shear Force Acting on Beam****1. Load Modifiers**

Load modifiers for beam design are shown in the flexure check.

**2. Strength I Limit State Load Factors**

Use the Strength I Limit State to determine the required shear resistance of the beam.

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above result indicates that the design truck and lane load on an interior beam control for shear.

**3. Strength I Limit State Shear Force**

Strength I Limit State factored shear force, two lanes loaded =  $V_{U(m)}$

**[Tables 3.4.1-1 and 3.4.1-2]**

$$V_{U(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot [V_{truck} + V_{lane}]]$$

$$V_{U(m)} = 1.0 \cdot [1.25 \cdot (5.18) + 1.50 \cdot (6.35) + 1.75 \cdot 1.0 \cdot [20.12]] = 51.21 \text{ kips}$$

**Check Shear Resistance of Beam****A. Factored Shear Resistance**

The factored shear force ( $V_{U(m)}$ ) is the required shear resistance of the beam that needs to be compared with the actual factored shear resistance of the beam ( $V_r$ ).

**[Eqns. 8.7-1, 8.7-2]**

For a rectangular wood section  $V_r = \phi_v \cdot F_v \cdot W_{bm} \cdot d_{bm} / 1.5$

**[8.5.2.2]****1. Resistance Factor**

Shear resistance factor =  $\phi_v = 0.75$

**2. Adjustment Factors for Reference Design Values****[8.4.4.2]**

Format conversion factor:  $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$

**[8.4.4.3]**

Wet Service factor =  $C_M = 0.875$

**[8.4.4.9]**

Time effect factor =  $C_\lambda = 0.80$

[Eqn. 8.4.4.1-2]

$$\begin{aligned} \text{Adjusted design value} &= F_v = F_{vx0} \cdot C_{KF} \cdot C_M \cdot C_\lambda \\ F_v &= 0.300 \cdot 3.33 \cdot 0.875 \cdot 0.80 = 0.699 \text{ ksi} \end{aligned}$$

**B. Beam Shear Check**

$$\text{Required beam shear resistance} = V_{U(m)}$$

For the beam to meet Strength I Limit State,  $V_{r(\text{prov})}$  must equal or exceed  $V_{U(m)}$ . As determined previously,  $V_{U(m)} = 51.21$  kips.

[Eqn. 8.7-2]

$$V_{r(\text{prov})} = \phi_v \cdot \frac{(F_v \cdot w_{bm} \cdot d_{bm})}{1.5} = 0.75 \cdot \frac{(0.699 \cdot 8.5 \cdot 46.75)}{1.5} = 138.88 \text{ kips}$$

$$V_{U(m)} = 51.21 \text{ kips} \leq V_{r(\text{prov})} = 138.88 \text{ kips}$$

OK

**Investigate  
Compression  
Resistance  
Requirements**

**A. Maximum Support Reactions per Beam****1. Dead Load Reaction Force**

The maximum shear/reactions were calculated above in the shear force check of the beam. The calculation below adds in the end diaphragm that was ignored in the shear calculation because it would normally be located within  $d_{\text{beam}}$  (depth of the component).

$$R_{dc\_max} = 2.90 + 2.81 + 0.18 + 0.59 + 0.362 = 6.84 \text{ kips}$$

$$R_{dw\_max} = 5.84 + 2.10 = 7.94 \text{ kips}$$

Maximum component dead load reaction force =  $R_{dc} = 6.84$  kips

Maximum wear course dead load reaction force =  $R_{dw} = 7.94$  kips

**2. Live Load Reactions**

The maximum live load reactions can be found in Table 3.4.1.2 of this Manual (Chapter 3).  $R_{\text{truck}}$  governs over  $R_{\text{tandem}}$ .

$$\text{The total reaction } R_{\text{Total}} = R_{\text{truck}} + R_{\text{lane}} = 56.0 + 13.40 = 69.4 \text{ kips}$$

For this example  $g_{\text{int}} = 0.60$  as calculated for flexure will be used. The distribution factor for shear was less than this and so is not used here. A minimum of half a design truck should typically be used. The 0.60 for flexure is larger than half a truck (or one wheel line) on one beam and so is sufficient in this case, and most similar cases. AASHTO LRFD does not provide live load distribution factors specifically for bearing of wood beams. The designer should evaluate axle load locations on the span for individual designs to make certain that the distribution factor used in design adequately determines the reaction on the bearing.

$$R_{LL} = 69.4 \cdot (0.60) = 41.64 \text{ kips}$$

**[3.4.1]** **B. Factored Support Reaction Forces Acting on Beam**  
Strength I Limit State maximum factored support reaction due to two lanes loaded case =  $P_{u(m)}$

**[Tables 3.4.1-1 and 3.4.1-2]**  $P_{u(m)} = \eta \cdot [1.25 \cdot R_{dc} + 1.50 \cdot R_{dw} + 1.75 \cdot r \cdot (R_{truck} + R_{lane})]$

$$P_{u(m)} = 1.0 \cdot [1.25 \cdot (6.84) + 1.50 \cdot (7.94) + 1.75 \cdot 1.0 \cdot (41.64)] = 93.33 \text{ kips}$$

**Check Compression Resistance of Beam**

**A. Factored Bearing Resistance**

The maximum factored support reaction  $P_{u(m)}$  is the required compression resistance perpendicular to the grain of the beam that needs to be compared with the actual factored compression resistance perpendicular to the grain of the beam ( $P_r$ ).

**[Eqns. 8.8.1-1, 8.8.3-1]**

$$P_r = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b$$

**[8.5.2.2]**

**1. Resistance Factor**

Compression perpendicular to grain resistance factor =  $\phi_{cperp} = 0.90$

**2. Adjustment Factors for Reference Design Values**

**[8.4.4.2]** Format conversion factor:  $C_{KF} = 2.1/\phi = 2.1/0.90 = 2.33$

**[8.4.4.3]** Wet Service factor =  $C_M = 0.53$

**[8.4.4.9]** Time effect factor =  $C_\lambda = 0.80$

**[Eqn. 8.4.4.1-5]**

Adjusted design value =  $F_{cp} = F_{cpo} \cdot C_{KF} \cdot C_M \cdot C_\lambda$

$$F_{cp} = 0.740 \cdot 2.33 \cdot 0.53 \cdot 0.80 = 0.731 \text{ ksi}$$

**3. Beam Bearing Dimensions**

For this calculation, a bearing length,  $L_b$ , of 18 inches will be tried.

Bearing width =  $b_b = w_{beam} = 8.5 \text{ in}$

$$\text{Bearing Area} = A_b = L_b \times b_b = 18.0 \times 8.5 = 153.0 \text{ in}^2$$

**[Table 8.8.3-1]**

**4. Bearing Adjustment Factor**

Adjustment Factor for Bearing =  $C_b$

$$L_b = 18.0 \text{ in} \geq 6.0 \text{ in} \quad C_b = 1.00$$

**B. Beam Bearing Resistance Check**

Required beam compression resistance =  $P_{u(m)} = 93.33 \text{ kips}$

For the beam to meet Strength I Limit State, provided compression resistance perpendicular to grain =  $P_{r(prov)}$  must equal or exceed  $P_{u(m)}$ .

**[Eqn. 8.8.3-1]**

$$P_{r(prov)} = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b = 0.9 \cdot 0.731 \cdot 153.0 \cdot 1.0 = 100.66 \text{ kips}$$

$$P_{u(m)} = 93.33 \text{ kips} \leq P_{r(prov)} = 100.66 \text{ kips}$$

OK

As stated at the beginning of Article 8.7.3, the bearing pad design is not a part of this example, so it will be assumed that the compression in the wood governs the bearing area size.

**8.7.4 Transverse  
Deck Design  
Examples**

The transverse deck design examples presented here go through the design of two wood deck types that can be used on top of the glulam beams designed in Article 8.7.3. Either of these deck types, transverse spike laminated or transverse glued laminated, could be used on the glulam beams to support the road surface. The final selection is up to the owner and designer, and might be influenced by availability and cost. If cost is the main determining factor, the final decision on type can be made after a design is done for each to determine which is most economical. Both of these deck types are available and used in Minnesota.

**[9.9.2]**

AASHTO LRFD Section 9 covers requirements for Decks and Deck Systems, including wood decks in 9.9. The nominal thickness of wood decks other than plank decks shall not be less than 6.0 in.

**[9.9.4.3.2]**

AASHTO LRFD requires a wear course on wood decks, and recommends bituminous. To prevent continual cracking of the bituminous and constant maintenance, bridge decks should consist of interconnected deck panels. Various options exist for connecting panels, but for these examples the panels are attached to each other using vertical spikes through ship lap joints along with longitudinal stiffener beams also called spreader beams. The deck panel depth and spreader beam sizes are based on deflection limits as well as strength considerations. The spreader beams enable the deck to act as a single unit under deflection and to consider it designed as interconnected in accordance with AASHTO LRFD.

**[9.9.4.2]**

Proper deck tie downs are important for a positive connection to the support for the deck, and to prevent excessive deflections that can occur when the deck is not securely fastened to each support. In the case of the transverse decks here, the timber beams are the supports. It is recommended to attach the deck to the beams with lag screws to stabilize the deck and prevent excess cracking in the bituminous wear course. The designer should determine lag bolt spacing for specific applications, but as a guide they are commonly spaced at 2 feet in the direction of the beams. In these examples the bituminous tapers down to 2 inches minimum, and so in this case the lag screw heads should be countersunk into the deck. It is best to shop drill and countersink, so that the panel wood is treated after countersinking. The wide beams in this example provide some tolerance for assembly on the beams in the field.

**[4.6.2.1.1]**

The deck span under investigation is an "equivalent" strip which spans from one beam to another beam. The deck overhang outside of the exterior beam should always be investigated. The deck cantilever does not need a complete analysis in this example because the exterior glulam

beams in Article 8.7.3 were positioned so that the deck overhang would not govern the deck design. Applying AASHTO LRFD 3.6.1.3.1 to this case, a wheel load along the curb will occur directly over the exterior beam, and not on the deck overhang.

**Transverse Spike  
Laminated Deck  
[9.9.6]**

**[Figure 8.3-1]**

**A. Material and Design Parameters**

The dimension annotations used throughout this design example are similar to a longitudinal deck. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively, considering the length to be in the direction transverse to the road centerline for a transverse deck. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications*, except for sawn lumber descriptive names. The letter notations will be used in this example (b, d, etc.).

Nominal dimensions for sawn lumber are always used for dead load calculations.

**1. Supporting Beams**

Length of the supporting members (bearing lengths for the deck on the beams) =  $b_{\text{length}} = 8.5$  in, determined in the previous example.

**[8.4.1.2.2]**

For glulam beams, the timber dimensions stated shall be taken as the actual net dimensions.

**[9.9.8]**

**2. Bituminous Wearing Surface**

MnDOT uses a 2% cross slope whenever practicable. In this case, minimum 2 inches at edge of roadway (face of curb) produces 6 inches at centerline. Because the deck spans are short, the thickness occurring within the span is used (not an average of the full deck width), and the largest force effect would be near the centerline of roadway. In addition, as described in Article 8.7.3 of this manual, the wearing surface will be thicker at the end of the deck due to beam camber. The thickness for deck design is then,  $d_{\text{WS}} = 6.9$  in.

**3. Curb and Railing [TL-4 Glulam Timber Rail with Curb]**

The timber barrier design is not a part of the design examples. The dimensions were used for weight considerations in Article 8.7.3. For this example, as described above, the deck overhang does not need to be analyzed and the curb and railing do not affect the deck spanning from beam to beam.

[8.4.1.1, 9.9.2]

#### 4. Deck Laminates

Assumed depth of timber deck panel laminates =  $d_{lam} = 5.75$  in

Assumed width of timber deck panel laminates =  $b_{lam} = 3.75$  in

[8.4.1.1.2]

Visually-graded transverse deck panel lumber is supplied rough sawn and typically surfaced on one side and one edge (S1S1E) to fabricate transverse deck panels to the specified dimensions. For nominal 4 in x 6 in lumber S1S1E reduces both the depth and width of an individual laminate by about 1/4 in. Nominal dimensions are used for dead loads, and surfaced dimensions are used in the section properties for strength.

[4.6.2.1.6]

#### 5. Span Lengths

In this case, MnDOT uses the effective span, or design span, as center to center of the deck bearing length on each beam, which is also center to center of beams, as stated in AASHTO LRFD.

Effective design span length for the deck panels =  $L_e = 5.0$  ft

#### 6. Unit Weights and Moisture Content

Type of deck panel wood material = Douglas Fir-Larch (No.2)

[Table 3.5.1-1]

Unit weight of soft-wood =  $\gamma_{DFL} = 0.050$  kcf

[MnDOT Table 3.3.1]

Unit weight of bituminous wearing surface =  $\gamma_{ws} = 0.150$  kcf

[MnDOT 3.3]

Standard MnDOT practice is to apply a future wearing course of 20 psf.

[8.4.1.1.3]

Moisture content (MC) of timber at the time of installation shall not exceed 19.0%

MnDOT designs for in-service wet-use only which is a MC of greater than 19% for sawn lumber.

[Table 8.4.1.1.4-1]

#### 7. Douglas Fir-Larch Deck (No. 2) Strength Properties

Reference Design Value for flexure =  $F_{bo} = 0.90$  ksi

Reference Design Value for compression perpendicular to grain  
=  $F_{cpo} = 0.625$  ksi

Reference Design Value for shear parallel to grain (horizontal)  
=  $F_{vo} = 0.18$  ksi

Modulus of elasticity =  $E_o = 1600$  ksi

#### Select the Basic Configuration

The bridge deck consists of interconnected deck panels, which are oriented perpendicular to traffic. The laminates of each panel will be connected using horizontal spikes. The panels are attached to each other using vertical spikes through ship lap joints along with longitudinal stiffener beams (also called spreader beams). The deck panel depth and

spreader beam sizes are based on deflection limits as well as strength considerations. The spreader beams enable the deck to act as a single unit under deflection, and to consider it interconnected by AASHTO LRFD. For a visual representation of the transverse deck on the glulam beams as well as the spreader beams, see Figure 8.7.3.1. The connections in the shiplap joints are similar to that shown in various figures in Article 8.7.1, except with a transverse deck the joints are also transverse as that is the direction of the panels.

#### A. Deck Panel Sizes

For shipping purposes, transverse deck panels are typically half the width of longitudinal panels. The dimensions of the panels at the beginning and end of deck are adjusted so that the total deck length matches the length of the beams.

The dimension lumber used in transverse decks typically needs to be spliced because of the longer lengths for the smaller cross-sectional sizes. Splices should be laid out to occur over interior beams, but splices should not occur in consecutive planks. The splices should be spaced every third or fourth plank.

#### B. Spreader Beam Dimensions

**[9.9.4.3.2]**

Interconnection of panels may be made with mechanical fasteners, splines, dowels, or stiffener beams. This example will use stiffener beams, or spreader beams, along with shiplap joints similar to the longitudinal deck in Article 8.7.1. For a transverse deck, the spreader beam is to be placed longitudinally along the bridge at the center of each deck span. The following rough sawn spreader beam dimensions will be verified.

Width of spreader beams =  $b_{spdr} = 5$  in

Depth of spreader beams =  $d_{spdr} = 5$  in

**[9.9.4.3]**

Minimum allowed rigidity of the spreader beams =  $EI_{min} = 80,000 \text{ kip}\cdot\text{in}^2$

Required moment of inertia of spreader beams to accommodate the specified rigidity for a given species and grade of wood =  $I_{min}$ .

For Douglas Fir-Larch No. 1 Posts & Timber,  $E_o = 1600$  ksi

Adjusted spreader beam modulus of elasticity =  $E$

**[8.4.4.3]**

**[Table 8.4.4.3-1]**

Wet Service factor for Modulus of Elasticity =  $C_M$

For nominal thickness > 4.0 in,  $C_M = 1.0$

[Eqn. 8.4.4.1-6]

Adjusted design value =  $E = E_o \times C_M$ 

$$E = 1600 \text{ ksi} \times 1.0 = 1600 \text{ ksi}$$

$$I_{\min} = \frac{80,000}{E} = \frac{80,000}{1600} = 50.0 \text{ in}^4$$

Check spreader beam dimensions.

$$I_{\text{spdr}} = \frac{1}{12} \cdot b_{\text{spdr}} \cdot d_{\text{spdr}}^3$$

$$I_{\text{spdr}} = \frac{1}{12} \cdot 5 \cdot 5^3 = 52.1 \text{ in}^4 \geq I_{\min} = 50.0 \text{ in}^4 \quad (\text{OK})$$

**Determine Dead and Live Load Reactions, Shear Forces, and Bending Moments**

The dead and live load shear, reaction and bending moment results can be determined using a basic structural analysis computer program, or using the standard beam formulas found in AISC 14<sup>th</sup> Edition LRFD Manual. MnDOT uses simplified analysis models that are permitted by AASHTO LRFD.

[4.6.2.1.6]

In the calculation of force effects using equivalent strips, the axle wheel loads may be considered point loads or patch loads, and the beams considered simply supported or continuous, as appropriate.

Modelling the axle wheel loads as patch loads will not have a large effect with the given beam spacing, and so for the calculations below the wheel loads on the axles are conservatively modelled as point loads.

[3.6.1.3.3]

Per AASHTO LRFD the design load in the design of decks is always an axle load; single wheel loads should not be considered. In addition, when using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the axles for the design tandem (whichever results in the largest effect) shall be applied to deck in determining live load force effects.

**A. Analysis Models**

In determining the maximum deck forces, MnDOT uses a variation of beam models for the deck strip as follows:

- 1) The maximum shear forces and reactions are determined by modeling the deck as a continuous beam. Moving live loads are then placed at various locations along the span, to produce the maximum shear and reactions. This method of analysis allows the effects of adjacent spans to be investigated. A two span continuous beam is conservatively assumed for simplicity.

- 2) The maximum positive bending moments (tension on deck bottom) and deflections are determined by considering the deck as a single simply-supported span between beams.
- 3) The maximum negative bending moments (tension on deck top) are determined by considering the deck as a single fixed-fixed span between beams, with fixed ends. Looking at the beam formulas in AISC 14<sup>th</sup> Edition LRFD Manual, it can be seen that this case will not govern, and so it will not be calculated here.

### B. Dead Loads per Unit Strip (1 ft)

The units for the dead load results are given in kips for a 1 ft wide transverse strip.

**1. Dead Loads per foot** (these units could also be given as kips per square foot).

$$\text{Weight of deck} = w_{\text{deck}} = \gamma_{\text{DFL}} \cdot d_{\text{lam}} = 0.050 \cdot 6/12 = 0.025 \text{ klf/ft}$$

$$\begin{aligned} \text{Weight of wearing course} &= w_{\text{ws}} = \gamma_{\text{ws}} \cdot d_{\text{ws}} \\ \gamma_{\text{ws}} \cdot d_{\text{ws}} &= 0.150 \cdot 6.9/12 = 0.086 \text{ klf/ft} \end{aligned}$$

$$\text{Weight of future wearing course} = w_{\text{FWC}} = 0.020 \text{ klf/ft}$$

### 2. Spreader beam point loads on 1 ft wide strip.

$$\text{Area of spreader beam} = A_{\text{spdr}} = d_{\text{spdr}} \cdot b_{\text{spdr}} = (5 \cdot 5)/144 = 0.174 \text{ ft}^2$$

$$\text{Spreader beam load} = P_{\text{spdr}} = \gamma_{\text{DFL}} \cdot A_{\text{spdr}} = 0.050 \cdot 0.174 = 0.009 \text{ kips/ft}$$

[AISC 14<sup>th</sup> p. 3-213]

### C. Dead Load Bending Moments per Unit Strip (1 ft)

Maximum bending moment due to deck weight =  $M_{\text{deck}}$

$$M_{\text{deck}} = \frac{w_{\text{deck}} \cdot (L_e)^2}{8} = \frac{0.025 \cdot 5.0^2}{8} = 0.078 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to wearing surface weight =  $M_{\text{ws}}$

$$M_{\text{ws}} = \frac{w_{\text{ws}} \cdot (L_e)^2}{8} = \frac{0.086 \cdot 5.0^2}{8} = 0.269 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to future wearing surface weight =  $M_{\text{FWC}}$

$$M_{\text{FWC}} = \frac{w_{\text{FWC}} \cdot (L_e)^2}{8} = \frac{0.020 \cdot 5.0^2}{8} = 0.063 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to spreader beam weight =  $M_{\text{spdr}}$

$$M_{\text{spdr}} = \frac{P_{\text{spdr}} \cdot L_e}{4} = \frac{0.009 \cdot 5.0}{4} = 0.011 \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to bridge component dead loads =  $M_{dc}$

$$M_{dc} = M_{deck} + M_{spdr}$$

$$M_{dc} = 0.078 + 0.011 = 0.089 \text{ kip}\cdot\text{ft}/\text{ft}$$

Maximum bending moments due to wearing course loads =  $M_{dw}$

$$M_{dw} = M_{ws} + M_{FWC}$$

$$M_{dw} = 0.269 + 0.063 = 0.332 \text{ kip}\cdot\text{ft}/\text{ft}$$

### [3.6.1.2]

#### D. Live Load Moments per Axle

The live load bending moment will be calculated per axle and later converted to a per unit strip (1 ft) format.

### [3.6.1.2.2]

#### 1. Design Truck Axle Loads

Point load on one deck span from design truck axle =  $P_{truck} = 16$  kips

Maximum bending moment due to design truck axle load =  $M_{truck}$

$$M_{truck} = \frac{P_{truck} \cdot L_e}{4} = \frac{16.0 \cdot 5.0}{4} = 20.000 \text{ kip}\cdot\text{ft}$$

### [3.6.1.2.3]

#### 2. Design Tandem Axle Loads

Point load of design tandem axle, one deck span =  $P_{tandem} = 12.5$  kips

AASHTO Table A4-1 can be used in the design of concrete decks, but includes impact so is not applicable to timber. However, the table footnotes indicate that specifically calculating the tandem is not necessary. A calculation can be done that shows the heavier single wheel load from the design truck on the smaller area of deck is the controlling case. Therefore, the tandem effect is not calculated for this example.

### [4.6.2.1]

#### E. Modification of Live Load Bending Moment

##### 1. Convert Live Load Bending Moment to Per Unit Strip

The live load bending moment calculated above ( $M_{truck}$ ) will now be distributed over the transverse equivalent strip width, and converted to a per foot basis.

### [Table 4.6.2.1.3-1]

For a structural deck thickness  $h = 5.75$  in, the equivalent strip width =  $E_s$   
 $= 4.0h + 40.0 = 63.0$  in

$$M_{truck} = M_{truck} \cdot \frac{1}{E_s} = 20.000 \cdot \frac{12}{63.0} = 3.810 \text{ kip}\cdot\text{ft}$$

### [3.6.1.1.2, 4.6.2.1]

#### 2. Multiple Presence Factors

The multiple presence factor is to be used in conjunction with the equivalent strip widths of 4.6.2.1.

**[3.6.1.1.1]**

Maximum number of traffic lanes on the deck =  $N_L$

$$N_L = \frac{b_{rd}}{12 \frac{\text{ft}}{\text{lane}}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

**[Table 3.6.1.1.2-1]**

For one lane loaded, the multiple presence factor =  $m = 1.20$

For two lanes loaded, the multiple presence factor =  $m = 1.00$

**[C3.6.1.1.2]**

This design example is for an unspecified ADTT, although AASHTO LRFD recommends limitations on the use of wood deck types based on ADTT. If these recommendations are adhered to, AASHTO LRFD also allows reduction of force effects based on ADTT because the multiple presence factors were developed on the basis of an ADTT of 5000 trucks in one direction. A reduction of 5% to 10% may be applied if the ADTT is expected to be below specified limits during the life of the bridge. If the ADTT level is confirmed, the reduction may be applied subject to the judgment of the designer and approved by the State Bridge Design Engineer.

**[AISC 14<sup>th</sup> p. 3-223]****F. Shear Force and Support Reactions**

As described above, shear force and reactions are calculated conservatively assuming a two span continuous beam. Axle tire loads can transversely occur at a distance as short as 4 ft apart if in two separate lanes, and if the two lanes are centered on a beam the axle tire loads are then 2 ft either side of a beam. This 2 lane case will need to be checked against the one lane case.

**[3.6.1.2.1]****[3.6.1.3.1]**

The axle tire placement for the one lane and two lane cases are illustrated in Figures 8.7.4.1 and 8.7.4.2.

The results are converted to a per foot basis and shown in Table 8.7.4.1. The live load force effects are shown for one and two lanes, with the appropriate multiple presence factor,  $m$ , applied.

### G. Summary of Maximum Shear Force, Reaction and Bending Moment Results

**Table 8.7.4.1**

Unfactored Load Case	Maximum Positive Bending Moment (kip-ft/ft)	Maximum Shear Force (kips/ft)	Maximum Support Reaction (kips/ft)
Component Dead Load (DC)	0.089	0.084	0.169
Wearing Course Dead Load (DW)	0.332	0.331	0.663
Live Loads			
Design Truck (1 lane, m=1.20)	4.572	2.775	3.113
Design Truck (2 lane, m=1.00)	3.810	2.414	4.827

#### *Flexural Check of Deck Panel*

### H. Factored Bending Moment per Unit Strip (1 ft)

#### 1. Load Modifiers

Standard MnDOT Load Modifiers are summarized in Table 3.2.1 of this manual.

For timber bridges  $\eta_D = 1.0$ . MnDOT considers spike laminated decks to have a conventional level of redundancy and uses  $\eta_R = 1.0$ . This example bridge is assumed to have a design ADT of over 500 for  $\eta_T = 1.0$ .

[1.3.2]

#### 2. Strength I Limit State Load Factors

[3.4.1]

Use the Strength I Limit State to determine the required resistance for the deck panels.

[3.6.2.3]

Impact factor need not be applied to wood components.

[4.6.2.3]

Skew factor (bridge is not skewed thus 1.0) =  $r = 1.0$

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis indicated that the truck load controls the bending moment of the deck panels. Therefore, use the truck load in determining the critical live load bending moment acting on the deck panels.

[Tables 3.4.1-1  
and 3.4.1-2]

### 3. Strength I Limit State Bending Moment per Unit Strip (1 ft)

Factored bending moment for the one lane loaded case =  $M_{U(m)}$

$$M_{U(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot (M_{truck} + M_{lane})]$$

$$M_{U(m)} = 1.0 \cdot [1.25 \cdot 0.089 + 1.50 \cdot 0.332 + 1.75 \cdot 1.0 \cdot (4.572)] = 8.610 \frac{\text{kip-ft}}{\text{ft}}$$

**Check Flexural  
Resistance of Deck  
Panel**

#### A. Factored Flexural Resistance

The factored bending moment ( $M_{U(m)}$ ) is the required flexural resistance of the deck that needs to be compared with the actual factored flexural resistance of the deck panel ( $M_R$ ).

[8.6.2]

For a rectangular wood section  $M_R = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$ .

[8.5.2.2]

#### 1. Resistance Factor

Flexural resistance factor =  $\phi_f = 0.85$

[8.6.2]

#### 2. Stability Factor

Stability factor for sawn dimension lumber in flexure =  $C_L$   
Laminated deck planks are fully braced.  $C_L = 1.0$

[8.4.4.4]  
[Table 8.4.4.4-1]

#### 3. Adjustment Factors for Reference Design Value

Size effect factor for sawn dimension lumber in flexure =  $C_F$

$$d_{lam} = 6 \text{ in}$$

$$b_{lam} = 4 \text{ in}$$

$$C_F = 1.30$$

[8.4.4.2]

Format conversion factor for component in flexure =  $C_{KF}$

$$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$$

[8.4.4.3]  
[Table 8.4.4.3-1]

Wet Service factor for sawn dimension lumber in flexure =  $C_M$

$$\text{Check } F_{bo} \cdot C_F: 0.900 \cdot 1.30 = 1.17 > 1.15$$

$$C_M = 0.85$$

[8.4.4.7]

Incising Factor for dimension lumber in flexure =  $C_i$

Douglas Fir-Larch requires incising for penetration of treatment.

$$C_i = 0.80$$

[Table 8.4.4.7-1]

[8.4.4.8]  
[Table 8.4.4.8-1]

Deck factor for a spike-laminated deck in flexure =  $C_d$

$$C_d = 1.15$$

[8.4.4.9]  
[Table 8.4.4.9-1]

Time effect factor for Strength I Limit State =  $C_\lambda$

$$C_\lambda = 0.80$$

[Eqn. 8.4.4.1-1]

$$\begin{aligned} \text{Adjusted design value} &= F_b = F_{b0} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_i \cdot C_d \cdot C_\lambda \\ F_b &= 0.900 \times 2.94 \times 0.85 \times 1.30 \times 0.80 \times 1.15 \times 0.80 = 2.152 \text{ ksi} \end{aligned}$$

**4. Required Section Modulus**

The section modulus is dependent on the deck panel depth. The section modulus is used in Part B to solve for the deck panel depth.

**B. Required Deck Panel Depth**

$$\text{Required deck flexural resistance} = M_{n(\text{req})}$$

For the deck panel depth to meet Strength I Limit State,  $M_r$  must equal (or exceed)  $M_{u(m)}$ , where  $M_r = \phi M_{n(\text{req})}$ . Therefore, set  $\phi M_{n(\text{req})} = M_{u(m)}$ .

$$M_{n(\text{req})} = \frac{M_{u(m)}}{\phi_f} = \frac{8.610}{0.85} = 10.129 \text{ kip-ft}$$

$$\text{Required Section Modulus of one foot of deck width} = S_{\text{req}}$$

$$\text{Required depth of deck laminates (panel)} = d_{\text{req}}$$

$$S_{\text{req}} = \frac{12 \text{ in} \cdot d_{\text{req}}^2}{6}$$

$$M_{n(\text{req})} = F_b \cdot S_{\text{req}} \cdot C_L \text{ with } C_L = 1.0$$

Substituting terms gives

$$d_{\text{req}} = \sqrt{\frac{6 \cdot M_{n(\text{req})}}{12 \cdot F_b \cdot C_L}} = \sqrt{\frac{6 \cdot 10.129 \cdot 12}{12 \cdot 2.152 \cdot 1.0}} = 5.31 \text{ in} \leq 5.75 \text{ in} \quad \text{OK}$$

The required deck panel depth (5.31 inches) indicates that the originally assumed deck depth (5.75 inches actual) can be used based on flexure. However, it is not uncommon that a deeper section could be required to satisfy the shear requirement, so that is checked next.

**Investigate Shear  
Resistance  
Requirements for  
Deck Panel  
[8.7, 9.9.3.2]**

**A. Critical Shear Force Location**

In transverse decks, maximum shear shall be computed at a distance from the support equal to the depth of the deck ( $d_{\text{lam}}$ ). The tire footprint shall be located adjacent to, and on the span side of, the point on the span where maximum force effect is sought.

$$\begin{aligned} \text{Location to check for shear} &= (d_{\text{lam}} + \frac{1}{2} \cdot b_{\text{length}}) / L_e \\ &= (0.48 \text{ ft} + \frac{1}{2} \cdot 0.71 \text{ ft}) / 5.0 \text{ ft} \end{aligned}$$

Check for shear at about 17% of span length away from the center of support, or 0.83 ft.

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical direction, but inherent vertical resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary.

**B. Unfactored Shear Acting on the Deck per Unit Strip (1 ft)**

For the uniformly distributed loads, the shear forces are less than the maximums listed in Table 8.7.4.1. The results given below are not the maximum shear forces on the deck (except for the design truck). Rather, they are the values taken at the appropriate distance "d<sub>lam</sub>" from the critical support face. The following shear forces were taken at the location 17% of span length from center support.

**1. Dead Load Shear Force**

Component dead load shear force at a distance "d<sub>lam</sub>" away from the support face = V<sub>dc</sub> = 0.059 kips

Wear course dead load shear force at a distance "d<sub>lam</sub>" away from the support face = V<sub>dw</sub> = 0.232 kips

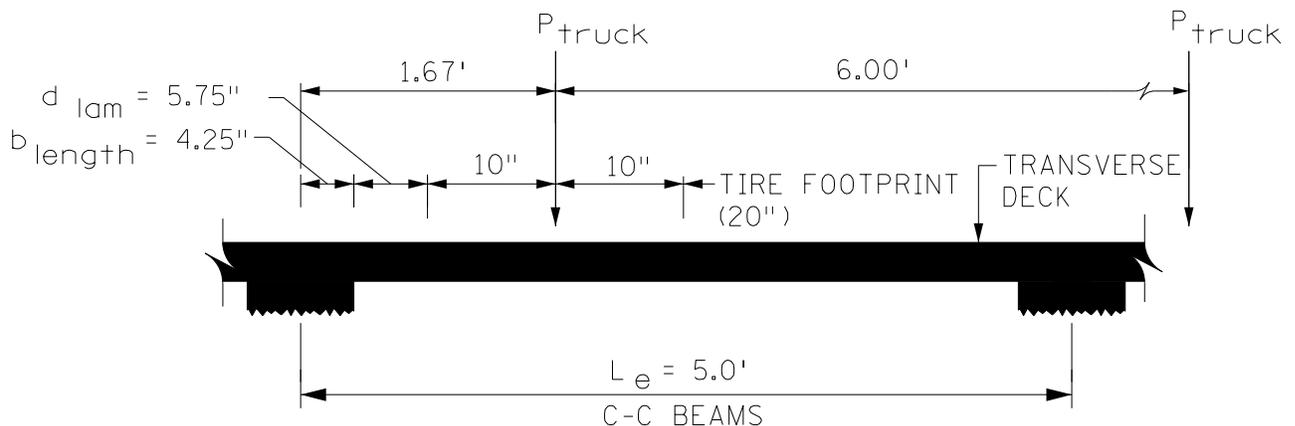
**2. Live Load Shear Forces**

Only the design truck is shown below. From the earlier results, this is the load case that gives the maximum shear force. One lane loaded with the multiple presence factor applied produces the maximum live load design shear forces as explained below.

**a. Design Truck Load One Lane Case**

[3.6.1.2.5]

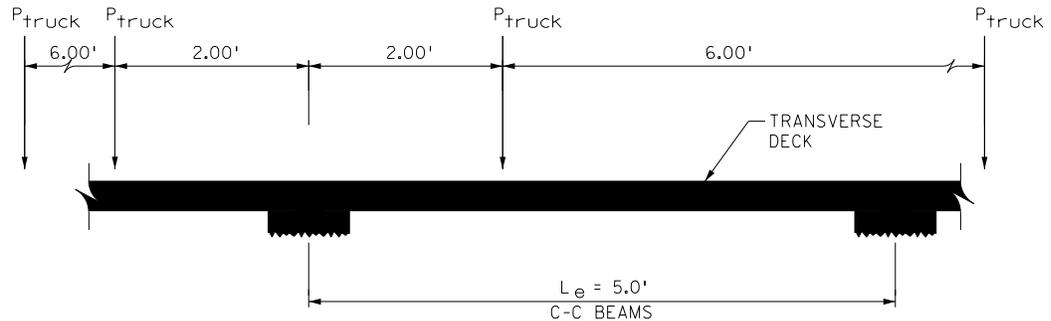
Truck tire contact area consists of a 20 inch width. Placing the 20 inch width according to 9.9.3.2 results in the following on one side of a support (beam) for the one lane case.



**Figure 8.7.4.1**

**[3.6.1.3.1]****b. Design Truck Load Two Lane Case**

For two adjacent loaded lanes, the closest another wheel can be placed on the opposite side of the support is 4.00 ft away, which is 2.33 ft from the support. If the minimum 4.00 ft space between wheels is centered on the support, the distance to the wheel on each side of the support is then 2.00 ft which satisfies the " $d_{lam}$ " minimum (1.67 ft), and is what produces the maximum force effects shown in Table 8.7.4.1.

**Figure 8.7.4.2**

Although the maximum calculated shear forces at a distance " $d_{lam}$ " away from the support for the design truck is governed by the case of two adjacent loaded lanes and is equal to the maximum  $= V_{truck} = 2.414$  kips, with the multiple presence factor applied the one lane loaded case governs the design shear as shown in Table 8.7.4.1.

**C. Factored Shear Acting on the Deck Panels per Unit Strip (1 ft)****1. Load Modifiers**

Load modifiers for deck design are shown in the flexure check.

**2. Strength I Limit State Load Factors**

Use the Strength I Limit State to determine the required shear resistance of the deck.

**[3.4.1]**

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results indicate that a single lane loaded with the design truck controls for shear.

**3. Strength I Limit State Shear Force**

Strength I Limit State factored shear force, one lane loaded  $= V_{u(m)}$

$$V_{u(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot [V_{truck(m)} + V_{lane(m)}]]$$

$$V_{u(m)} = 1.0 \cdot [1.25 \cdot (0.059) + 1.50 \cdot 0.232] + 1.75 \cdot 1.0 \cdot [2.775] = 5.28 \text{ kips}$$

**[Tables 3.4.1-1  
and 3.4.1-2]**

**Check Shear  
Resistance of Deck  
Panel**

**A. Factored Shear Resistance**

The factored shear force  $V_{u(m)}$  is the required shear resistance of the deck that needs to be compared with the actual factored shear resistance of the deck ( $V_r$ ).

[Eqns. 8.7-1, 8.7-2] For a rectangular wood section  $V_r = \phi_v \cdot F_v \cdot b \cdot d_{lam} / 1.5$

[8.5.2.2]

**1. Resistance Factor**

Shear resistance factor =  $\phi_v = 0.75$

[8.4.4.2]

**2. Adjustment Factors for Reference Design Values**

Format conversion factor:  $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$

[8.4.4.3]

Wet Service factor =  $C_M = 0.97$

[8.4.4.7]

Incising Factor for dimension lumber in flexure ( $F_{bo}$ ) =  $C_i$

Douglas Fir-Larch requires incising for penetration of treatment.

[Table 8.4.4.7-1]

$C_i = 0.80$

[8.4.4.9]

Time effect factor =  $C_\lambda = 0.80$

[Eqn. 8.4.4.1-2]

Adjusted design value =  $F_v = F_{v0} \cdot C_{KF} \cdot C_M \cdot C_i \cdot C_\lambda$

$F_v = 0.18 \cdot 3.33 \cdot 0.97 \cdot 0.80 \cdot 0.80 = 0.372$  ksi

**B. Deck Panel Shear Check**

Required deck shear resistance =  $V_{u(m)}$

For the deck to meet Strength I Limit State,  $V_{r(prov)}$  must equal or exceed  $V_{u(m)}$ . As determined previously,  $V_{u(m)} = 5.28$  kips.

[Eqn. 8.7-2]

$$V_{r(prov)} = \phi_v \cdot \frac{(F_v \cdot b \cdot d_{lam})}{1.5} = 0.75 \cdot \frac{(0.372 \cdot 12 \cdot 5.75)}{1.5} = 12.83 \text{ kips}$$

$$V_{u(m)} = 5.28 \text{ kips} \leq V_{r(prov)} = 12.83 \text{ kips}$$

OK

**Check  
Compression  
Resistance**

Compression, or bearing of the deck on the beams, should be computed in accordance with the provisions of AASHTO LRFD for non-standard situations that provide a very narrow bearing area for the transverse deck. For this example, compression bearing on the glued laminated beams is not close to governing the design of the deck panel and so the calculation is not shown here. It usually will not govern a transverse deck design for a bridge of standard configuration. A bearing resistance calculation check for the longitudinal deck (on the pier caps) is shown in Article 8.7.1.

**Investigate  
Deflection  
Requirements  
[9.9.3.3]**

**A. Deck Live Load Deflection with Current Deck Parameters**

The final check for the transverse deck design to meet AASHTO LRFD is the deformation, or deflection, calculation. The design truck will have the most severe effect, and that is used for checking the transverse deck deflection.

**[3.6.1.3.3]**

When using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the design tandem (whichever results in the largest effect) shall be applied to the deck in determining live load force effects.

Deflections are to be calculated using Service I Limit State.

Calculate deck deflections for a transverse interconnected deck using a per foot width approach. This approach can be used on a spike laminated deck with shiplap joints and a spreader beam.

**[2.5.2.6.2]**

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

**[C2.5.2.6.2]**

As of note, if a plank deck or a non-interconnected panel deck is being analyzed, a different approach likely is required for the live load distribution, and an additional limitation of 0.10 inches relative deflection between adjacent edges is also required.

**1. Deck Stiffness**

Moment of inertia of one foot width of deck panels =  $I_{prov}$

$$I_{prov} = \frac{1}{12} \cdot b \cdot d_{lam}^3 = \frac{1}{12} \cdot 12 \cdot (5.75)^3 = 190 \text{ in}^4$$

Adjusted deck panel modulus of elasticity = E

Wet Service Factor for Modulus of Elasticity =  $C_M$

$$C_M = 0.90$$

Incising Factor for Modulus of Elasticity =  $C_i$

$$C_i = 0.95$$

**[8.4.4.3]**

**[Table 8.4.4.3-1]**

**[Table 8.4.4.7-1]**

**[Eqn. 8.4.4.1-6]**

$$E = E_o \cdot C_M \cdot C_i = 1600 \text{ ksi} \times 0.90 \times 0.95 = 1368.0 \text{ ksi}$$

## 2. Loads per Unit Strip Width (1 ft)

Design truck load on deck span used for deflection calculations =  $P_{\Delta\text{truck}}$ .

Similar to calculations for the maximum positive bending moments, deflections are determined by considering the deck as a single simply-supported span between beams. Therefore, the point load on one deck span from design truck axle =  $P_{\text{truck}} = 16$  kips.

$P_{\text{truck}}$  expressed as per foot width =  $P_{\Delta\text{truck}}$ :

$$P_{\Delta\text{truck}} = P_{\text{truck}} \cdot 12 \text{ in} / E_s = P_{\text{truck}} \cdot 12 \text{ in} / 63 \text{ in}$$

$$P_{\Delta\text{truck}} = 16 \cdot 0.191 = 3.05 \text{ kips/ft}$$

**[Table 3.6.1.1.2-1]**

One lane loaded governs, the multiple presence factor =  $m = 1.20$

**[3.6.1.3]**

**[AISC 14<sup>th</sup> p. 3-215]**

## 3. Live Load Deflection Calculations

Deflection at deck midspan due to design truck load axle load =  $\Delta_{\text{truck}}$

$$\Delta_{\text{truck}} = \frac{m \cdot P_{\Delta\text{truck}} \cdot L_e^3}{48 \cdot E \cdot I_{\text{prov}}} = \frac{1.20 \cdot 3.05 \cdot (5.00 \cdot 12)^3}{48 \cdot 1368.0 \cdot 190} = 0.06 \text{ in}$$

The maximum deflection live load deflection =  $\Delta_{\text{truck}} = 0.06$  in

**[2.5.2.6.2]**

Live load deflection limit at deck midspan =  $\Delta_{\text{max}}$

$$\Delta_{\text{max}} = L_e / 425 = 5.0 \text{ ft} \cdot 12 \text{ in} / 425 \cdot \text{ft} = 0.14 \text{ in}$$

$$\Delta = 0.06 \text{ in} \leq \Delta_{\text{max}} = 0.14 \text{ in}$$

OK

Deflections are also okay. Thus, the initial 6 inch nominal deck panel depth and grade are adequate for the design.

**Transverse Glued**

**Laminated Deck**

**[9.9.4]**

## A. Material and Design Parameters

The dimension annotations used throughout this design example are similar to that for the transverse spike laminated deck. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively, considering the length to be in the direction transverse to the road centerline for a transverse deck. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications* letter notations for sawn lumber (but not the descriptive names). The glulam definitions in Figure 8.3-1 are set up for a glulam beam, and are not applicable to a transverse glulam deck panel. The sawn lumber letter notations will be used in this example (b, d, etc.).

**[Figure 8.3-1]**

**[8.4.1.2.2]**

Dimensions stated for glued laminated timber shall be taken as the actual net dimensions.

**1. Supporting Beams**

Length of the supporting members (bearing lengths for the deck on the beams) =  $b_{\text{length}} = 8.5$  in, determined in the beam design example. The dimensions stated shall be taken as the actual net dimensions.

**[9.9.8]****2. Bituminous Wearing Course**

MnDOT uses a 2% cross slope whenever practicable. In this case, minimum 2 in at edge of roadway (face of curb) produces 6 in at centerline. Because the deck spans are short, the thickness occurring within the span is used (not an average of the full deck width), and the largest force effect would be near the centerline of roadway. In addition, the wearing surface will be thicker at the end of the deck due to beam camber. The thickness for deck design is then,  $d_{\text{ws}} = 6.9$  in.

**3. Curb and Railing [TL-4 Glulam Timber Rail with Curb]**

The timber barrier design is not a part of the design examples. The dimensions were used for weight considerations in Article 8.7.3. For the deck examples, as described above, the deck overhang does not need to be analyzed and the curb and railing do not affect the deck spanning from beam to beam.

**[8.4.1.2.2, 9.9.2]****4. Glulam Deck Panels, Southern Pine**

Assumed depth of timber deck panel laminates =  $d_{\text{lam}} = 5.00$  in

Assumed width of timber deck panel laminates =  $b_{\text{lam}} = 1.375$  in

Attention must be given to the species of wood, as laminate widths and thicknesses vary by species. For a nominal 6 inch wide lamination in Southern Pine, a net finished width of 5 inches or 5 1/8 inches is available (which is the deck depth with the glulam placed flatwise).

Because the individual laminates in the glued laminated deck panels are not orientated horizontally as in a beam, the glulam combinations generally intended for axial loading are commonly used for transverse decks, instead of the combinations normally used for beams.

**[4.6.2.1.6]****5. Span Lengths**

In this case, MnDOT uses the effective span, or design span, as center to center of the deck bearing length on each beam, which is also center to center of beams, as stated in AASHTO LRFD.

Effective design span length for the deck panels =  $L_e = 5.0$  ft

## 6. Unit Weights and Moisture Content

Type of glulam panel wood material = Southern Pine (ID No. 48)

**[Table 3.5.1-1]**

Unit weight of soft-wood =  $\gamma_{SP} = 0.050$  kcf

**[MnDOT Table 3.3.1]**

Unit weight of bituminous wearing surface =  $\gamma_{WS} = 0.150$  kcf

**[MnDOT 3.3]**

Standard MnDOT practice is to apply a future wearing course of 20 psf.

**[8.4.1.1.3]**

Moisture content (MC) of timber at the time of installation shall not exceed 19.0%

MnDOT designs for in-service wet-use only which is a MC of greater than 16% for glulam.

## 7. Southern Pine Glulam Deck (ID No. 48) Strength Properties

**[Table 8.4.1.2.3-2]**

Reference Design Value for flexure =  $F_{bYO} = 2.000$  ksi

Reference Design Value for compression perpendicular to grain

=  $F_{CPO} = 0.740$  ksi

Reference Design Value for shear parallel to grain (horizontal shear)

=  $F_{VYO} = .260$  ksi

Modulus of elasticity =  $E_o = 1700$  ksi

### Select the Basic Configuration

The bridge deck consists of interconnected deck panels, which are oriented perpendicular to traffic. The panels are manufactured using wet use adhesives to join the individual laminates into panels. The panels are attached to each other using vertical spikes through ship lap joints along with longitudinal stiffener beams also called spreader beams. The deck panel depth and spreader beam sizes are based on deflection limits as well as strength considerations. The spreader beams enable the deck to act as a single unit under deflection and to consider it interconnected in accordance with AASHTO LRFD.

For a visual representation of the transverse deck on the glulam beams as well as the spreader beams, Figure 8.7.3.1 of this manual can be referenced. The connections in the shiplap joints are similar to that shown in various figures in Article 8.7.1, except with a transverse deck the joints are also transverse as that is the direction of the panels.

### A. Deck Panel Sizes

Transverse glulam deck panels vary in width between 3.0 and 6.0 feet. The dimensions of the panels at the beginning and end of deck are adjusted so that the total deck length matches the length of the beams. The panels are to be manufactured meeting the requirements of ANSI/AITC A190.1. The panels are required to be manufactured using wet use adhesives to join the individual laminates to attain the specified

panel size, and under this condition the adhesive bond is stronger than the wood laminates.

### **B. Spreader Beam Dimensions**

#### **[9.9.4.3.2]**

Interconnection of panels may be made with mechanical fasteners, splines, dowels, or stiffener beams. This example will use stiffener beams, or spreader beams, along with shiplap joints similar to the transverse spike laminated deck. For a transverse deck, the spreader beam is to be placed longitudinally along the bridge at the center of each deck span.

Glulam panels are sometimes designed with horizontal dowel connections which can be effective for transferring loads between panels under ideal conditions, but in practice can be difficult to construct properly. The shiplap joint and spreader beam eliminates the field fit up and installation problems associated with the dowel connections.

The following rough sawn spreader beam dimensions that were verified in the Transverse Spike Laminated Deck Design Example will also be used in this design example (refer to that example for the calculation).

Width of spreader beams =  $b_{\text{spdr}} = 5$  in

Depth of spreader beams =  $d_{\text{spdr}} = 5$  in

If preferred by the designer, a similar sized glulam spreader beam could be checked and used in this design for a transverse glulam deck, provided it meets the minimum rigidity requirements.

#### **[9.9.4.3]**

The rigidity of the spreader beam shall be at least 80,000 kip-in<sup>2</sup>.

### **Determine Dead and Live Load Reactions, Shear Forces, and Bending Moments**

The dead and live load shear, reaction and bending moment results can be determined using a basic structural analysis computer program, or using the standard beam formulas found in AISC 14<sup>th</sup> Edition LRFD Manual. MnDOT uses simplified analysis models that are permitted by AASHTO LRFD.

#### **[4.6.2.1.6]**

In the calculation of force effects using equivalent strips, the axle wheel loads may be considered point loads or patch loads, and the beams considered simply supported or continuous, as appropriate.

Modelling the axle wheel loads as patch loads will not have a large effect with the given beam spacing, and so for the calculations below the wheel loads on the axles are conservatively modelled as point loads.

**[3.6.1.3.3]**

Per AASHTO LRFD the design load in the design of decks is always an axle load; single wheel loads should not be considered. In addition, when using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the axles for the design tandem (whichever results in the largest effect) shall be applied to deck in determining live load force effects.

**A. Analysis Models**

In determining the maximum deck forces, MnDOT uses a variation of beam models for the deck strip as follows:

- 1) The maximum shear forces and reactions are determined by modeling the deck as a continuous beam. Moving live loads are then placed at various locations along the span, to produce the maximum shear and reactions. This method of analysis allows the effects of adjacent spans to be investigated. A two span continuous beam is conservatively assumed for simplicity.
- 2) The maximum positive bending moments (tension on deck bottom) and deflections are determined by considering the deck as a single simply-supported span between beams.
- 3) The maximum negative bending moments (tension on deck top) are determined by considering the deck as a single fixed-fixed span between beams, with fixed ends. Looking at the beam formulas in AISC 14<sup>th</sup> Edition LRFD Manual, it can be seen that this case will not govern, and so it will not be calculated here.

**B. Dead Loads per Unit Strip (1 ft)**

The units for the dead load results are given in kips for a 1 ft wide transverse strip.

**1. Dead Loads per foot** (these units could also be given as kips per square foot).

$$\text{Weight of deck} = w_{\text{deck}} = \gamma_{\text{SP}} \cdot d_{\text{lam}} = 0.050 \cdot 5.0/12 = 0.021 \text{ klf/ft}$$

$$\text{Weight of wear course} = w_{\text{WS}} = \gamma_{\text{WS}} \cdot d_{\text{WS}} = 0.150 \cdot 6.9/12 = 0.086 \text{ klf/ft}$$

$$\text{Weight of future wearing course} = w_{\text{FWC}} = 0.020 \text{ klf/ft}$$

**2. Spreader beam point loads on 1 ft wide strip.**

$$\text{Area of spreader beam} = A_{\text{spdr}} = d_{\text{spdr}} \cdot b_{\text{spdr}} = (5 \cdot 5)/144 = 0.174 \text{ ft}^2$$

$$\text{Spreader beam load} = P_{\text{spdr}} = \gamma_{\text{DFL}} \cdot A_{\text{spdr}} = 0.050 \cdot 0.174 = 0.009 \text{ kips/ft}$$

**[AISC 14<sup>th</sup> p. 3-213] C. Dead Load Bending Moments per Unit Strip (1 ft)**

Maximum bending moment due to deck weight

$$M_{\text{deck}} = \frac{w_{\text{deck}} \cdot (L_e)^2}{8} = \frac{0.021 \cdot 5.0^2}{8} = 0.066 \frac{\text{kip}\cdot\text{ft}}{\text{ft}}$$

Maximum bending moment due to wearing surface weight

$$M_{\text{ws}} = \frac{w_{\text{ws}} \cdot (L_e)^2}{8} = \frac{0.086 \cdot 5.0^2}{8} = 0.269 \frac{\text{kip}\cdot\text{ft}}{\text{ft}}$$

Maximum bending moment due to future wearing surface weight

$$M_{\text{FWC}} = \frac{w_{\text{FWC}} \cdot (L_e)^2}{8} = \frac{0.020 \cdot 5.0^2}{8} = 0.063 \frac{\text{kip}\cdot\text{ft}}{\text{ft}}$$

Maximum bending moment due to spreader beam weight

$$M_{\text{spdr}} = \frac{P_{\text{spdr}} \cdot L_e}{4} = \frac{0.009 \cdot 5.0}{4} = 0.011 \frac{\text{kip}\cdot\text{ft}}{\text{ft}}$$

Maximum bending moment due to bridge component dead loads

$$M_{\text{dc}} = M_{\text{deck}} + M_{\text{spdr}} \quad M_{\text{dc}} = 0.066 + 0.011 = 0.077 \text{ kip}\cdot\text{ft}/\text{ft}$$

Maximum bending moments due to wearing course loads =  $M_{\text{dw}}$

$$M_{\text{dw}} = M_{\text{ws}} + M_{\text{FWC}} \quad M_{\text{dw}} = 0.269 + 0.063 = 0.332 \text{ kip}\cdot\text{ft}/\text{ft}$$

**[3.6.1.2]****D. Live Load Moments per Axle**

The live load bending moment will be calculated per wheel and later converted to a per unit strip (1 ft) format.

**[3.6.1.2.2]****1. Design Truck Axle Loads**

Point load on one deck span from design truck axle =  $P_{\text{truck}} = 16$  kips

Maximum bending moment due to design truck wheel load

$$M_{\text{truck}} = \frac{P_{\text{truck}} \cdot L_e}{4} = \frac{16.0 \cdot 5.0}{4} = 20.000 \text{ kip}\cdot\text{ft}$$

**[3.6.1.2.3]****2. Design Tandem Axle Loads**

Point load of design tandem wheel =  $P_{\text{tandem}} = 12.5$  kips

AASHTO Table A4-1 can be used in the design of concrete decks, but includes impact so is not applicable to timber. However, the table footnotes indicate that specifically calculating the tandem is not necessary. A calculation can be done that shows the heavier single wheel load from the design truck on the smaller area of deck is the controlling case. Therefore, the tandem effect is not calculated for this example.

**[4.6.2.1]****E. Modification of Live Load Bending Moment****1. Convert Live Load Bending Moment to Per Unit Strip**

The live load bending moment calculated above ( $M_{\text{truck}}$ ) will now be distributed over the transverse equivalent strip width, and converted to a per foot basis.

**[Table 4.6.2.1.3-1]**

For a structural deck thickness  $h = 5.0$  in, the equivalent strip width =  $4.0h + 30.0 = 50.0$  in

$$M_{\text{truck}} = M_{\text{truck}} \cdot \frac{1}{E_s} = 20.000 \cdot \frac{12}{50.0} = 4.800 \text{ kip-ft}$$

**2. Multiple Presence Factors****[3.6.1.1.2, 4.6.2.1]**

The multiple presence factor is to be used in conjunction with the equivalent strip widths of 4.6.2.1.

**[3.6.1.1.1]**

Maximum number of traffic lanes on the deck =  $N_L$

$$N_L = \frac{b_{\text{rd}}}{12 \frac{\text{ft}}{\text{lane}}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

**[Table 3.6.1.1.2-1]**

For one lane loaded, the multiple presence factor =  $m = 1.20$

For two lanes loaded, the multiple presence factor =  $m = 1.00$

**[C3.6.1.1.2]**

This design example is for an unspecified ADTT, although AASHTO LRFD recommends limitations on the use of wood deck types based on ADTT. If these recommendations are adhered to, AASHTO LRFD also allows reduction of force effects based on ADTT because the multiple presence factors were developed on the basis of an ADTT of 5000 trucks in one direction. A reduction of 5% to 10% may be applied if the ADTT is expected to be below specified limits during the life of the bridge. If the ADTT level is confirmed, the reduction may be applied subject to the judgment of the designer and approved by the State Bridge Design Engineer.

**[AISC 14<sup>th</sup> p. 3-223]****Shear Force and Support Reactions**

As described above, shear force and reactions are calculated conservatively assuming a two span continuous beam. Axle tire loads can occur transversely at a distance as short as 4 ft apart if in two separate lanes, and if the two lanes are centered on a beam the axle tire loads are then 2 ft either side of a beam. This 2 lane case will need to be checked against the one lane case.

**[3.6.1.2.1]****[3.6.1.3.1]**

The axle tire placement for the one lane and two lane cases are illustrated below with diagrams, which are shown under the Chapter section "Investigate Shear Resistance Requirements for Deck Panel".

The results are converted to a per foot basis and shown in the table below. The live load force effects are shown for one and two lanes, with the appropriate multiple presence factor,  $m$ , applied.

### G. Summary of Maximum Shear Force, Reaction and Bending Moment Results

**Table 8.7.4.2**

Unfactored Load Case	Maximum Positive Bending Moment (kip-ft/ft)	Maximum Shear Force (kips/ft)	Maximum Support Reaction (kips/ft)
Component Dead Load (DC)	0.077	0.071	0.143
Wearing Course Dead Load (DW)	0.332	0.331	0.663
Live Loads			
Design Truck (1 lane, $m=1.2$ )	5.760	3.555	3.976
Design Truck (2 lane, $m=1.0$ )	4.800	3.041	6.082

### Flexural Check of Deck Panel

#### [1.3.2]

### H. Factored Bending Moment per Unit Strip (1 ft)

#### 1. Load Modifiers

Standard MnDOT Load Modifiers are summarized in Table 3.2.1 of this manual.

For timber bridges  $\eta_D = 1.0$ . MnDOT considers glued laminated decks to have a conventional level of redundancy and uses  $\eta_R = 1.0$ . This example bridge is assumed to have a design ADT of over 500 for  $\eta_T = 1.0$ .

#### 2. Strength I Limit State Load Factors

#### [3.4.1]

Use the Strength I Limit State to determine the required resistance for the deck panels.

#### [3.6.2.3]

Impact factor need not be applied to wood components.

#### [4.6.2.3]

Skew factor (bridge is not skewed thus 1.0) =  $r = 1.0$

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis indicated that the truck load controls the bending moment of the deck panels. Therefore, use the truck load in determining the critical live load bending moment acting on the deck panels.

### 3. Strength I Limit State Bending Moment per Unit Strip (1 ft)

Factored bending moment for the one lane loaded case =  $M_{u(m)}$

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.5 \cdot M_{dw} + 1.75 \cdot r \cdot [M_{truck} + M_{lane}]]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot 0.077 + 1.50 \cdot 0.332 + 1.75 \cdot 1.0 \cdot [5.76]] = 10.674 \frac{\text{kip-ft}}{\text{ft}}$$

[Tables 3.4.1-1  
and 3.4.1-2]

#### Check Flexural Resistance of Deck Panel

#### A. Factored Flexural Resistance

The factored bending moment ( $M_{u(m)}$ ) is the required flexural resistance of the deck that needs to be compared with the actual factored flexural resistance of the deck panel ( $M_r$ ).

[8.6.2]

For a rectangular wood section  $M_r = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$ .

[8.5.2.2]

#### 1. Resistance Factor

Flexural resistance factor =  $\phi_f = 0.85$

[8.6.2]

#### 2. Stability Factor

Stability factor for glulam lumber in flexure =  $C_L$

Laminated deck planks are fully braced.  $C_L = 1.0$

[8.4.4.2]

#### 3. Adjustment Factors for Reference Design Value

Format conversion factor for component in flexure =  $C_{KF}$

$$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$$

[8.4.4.3]

Wet Service factor for glued laminated timber in flexure =  $C_M$

[Table 8.4.4.3-2]

$$C_M = 0.80$$

[8.4.4.6]

Flat use factor for vertically laminated glulam timber in flexure =  $C_{fu}$

[Table 8.4.4.6-2]

$$d_{lam} = 5.0 \text{ in}$$

$$C_{fu} = 1.10$$

[8.4.4.9]

Time effect factor for Strength I Limit State =  $C_\lambda$

[Table 8.4.4.9-1]

$$C_\lambda = 0.80$$

[Eqn. 8.4.4.1-1]

Adjusted design value =  $F_b = F_{by0} \cdot C_{KF} \cdot C_M \cdot C_{fu} \cdot C_\lambda$

$$F_b = 2.00 \times 2.94 \times 0.80 \times 1.10 \times 0.80 = 4.140 \text{ ksi}$$

#### 4. Required Section Modulus

The section modulus is dependent on the deck panel depth. The section modulus is used in Part B to solve for the deck panel depth.

#### B. Required Deck Panel Depth

Required deck flexural resistance =  $M_{n(\text{req})}$

For the deck panel depth to meet Strength I Limit State,  $M_r$  must equal (or exceed)  $M_{u(m)}$ , where  $M_r = \phi M_{n(\text{req})}$ . Therefore, set  $\phi M_{n(\text{req})} = M_{u(m)}$ .

$$M_{n(\text{req})} = \frac{M_{u(m)}}{\phi_f} = \frac{10.674}{0.85} = 12.558 \text{ kip} \cdot \text{ft}$$

Required section modulus of one foot of deck width =  $S_{\text{req}}$

Required depth of deck laminates (panel) =  $d_{\text{req}}$

$$S_{\text{req}} = \frac{12 \text{ in} \cdot d_{\text{req}}^2}{6}$$

$M_{n(\text{req})} = F_b \cdot S_{\text{req}} \cdot C_L$ , with  $C_L = 1.0$ , substituting terms gives

$$d_{\text{req}} = \sqrt{\frac{6 \cdot M_{n(\text{req})}}{12 \text{ in} \cdot F_b \cdot C_L}} = \sqrt{\frac{6 \cdot 12.558 \cdot 12}{12 \text{ in} \cdot 4.140 \cdot 1.0}} = 4.27 \text{ in} \leq 5.0 \text{ in} \quad \text{OK}$$

The required deck panel depth (4.27 inches) indicates that the originally assumed deck depth (5.0 inches) can be used based on flexure. However, it is not uncommon that a deeper section could be required to satisfy the shear requirement, so that is checked next.

#### **Investigate Shear Resistance Requirements for Deck Panel** [8.7, 9.9.3.2]

#### A. Critical Shear Force Location

In transverse decks, maximum shear shall be computed at a distance from the support equal to the depth of the deck ( $d_{\text{lam}}$ ). The tire footprint shall be located adjacent to, and on the span side of, the point of the span where maximum force effect is sought.

$$\begin{aligned} \text{Location to check for shear} &= (d_{\text{lam}} + \frac{1}{2} \cdot b_{\text{length}}) / L_e \\ &= (0.42 \text{ ft} + \frac{1}{2} \cdot 0.71 \text{ ft}) / 5.0 \text{ ft} \end{aligned}$$

Check for shear at about 16% of span length away from the center of support, or 0.78 ft.

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical

direction, but inherent resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary on this.

**B. Unfactored Shear Acting on the Deck per Unit Strip (1 ft)**

For the uniformly distributed loads, the shear forces are less than the maximums listed in the earlier table (Table 8.7.4.2). The results given below are not the maximum shear forces on the deck (except for the design truck). Rather, they are the values taken at the appropriate distance "d<sub>lam</sub>" from the critical support face. The following shear forces were taken at the location 16% of span length from center support.

**1. Dead Load Shear Force**

Component dead load shear force at a distance "d<sub>lam</sub>" away from the support face =  $V_{dc} = 0.051$  kips

Wear course dead load shear force at a distance "d<sub>lam</sub>" away from the support face =  $V_{dw} = 0.239$  kips

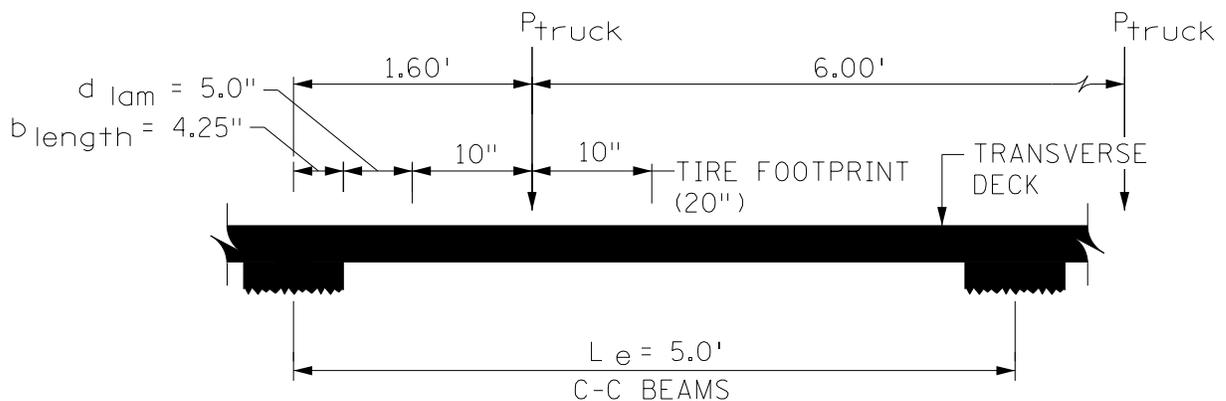
**2. Live Load Shear Forces**

Only the design truck is shown below. From the earlier results, this is the load case that gives the maximum shear force. One lane loaded with the multiple presence factor applied produces the maximum live load design shear forces as explained below.

**a. Design Truck Load One Lane Case**

[3.6.1.2.5]

Truck tire contact area consists of a 20 inch width. Placing the 20 inch width according to 9.9.3.2 results in the following on one side of a support (beam) for the one lane case.

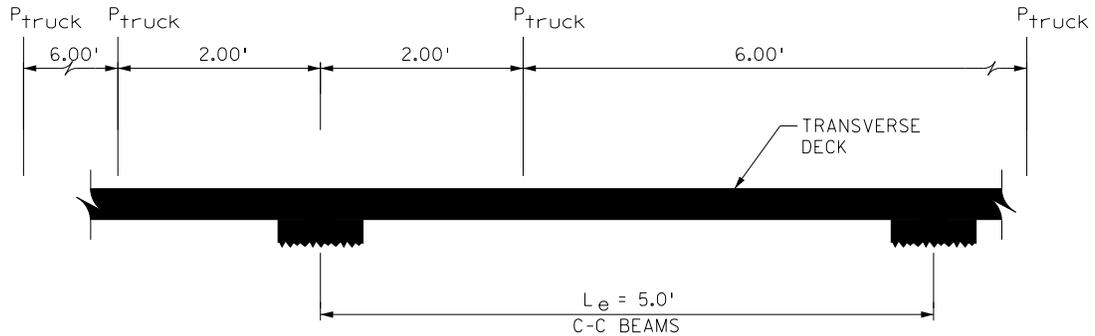


**Figure 8.7.4.3**

**b. Design Truck Load Two Lane Case**

[3.6.1.3.1]

For two adjacent loaded lanes, the closest another wheel can be placed on the opposite side of the support is 4.00 ft away, which is 2.40 ft from the support. If the minimum 4.00 ft space between wheels is centered on the support, the distance to the wheel on each side of the support is then 2.00 ft which satisfies the "d<sub>lam</sub>" minimum (1.60 ft), and is what produces the maximum force effects shown in Table 8.7.4.2.



**Figure 8.7.4.4**

Although the maximum calculated shear forces at a distance "d<sub>lam</sub>" away from the support for the design truck is governed by the case of two adjacent loaded lanes and is equal to the maximum = V<sub>truck</sub> = 3.041 kips, with the multiple presence factor applied the one lane loaded case governs the design shear as shown in Table 8.7.4.2.

**C. Factored Shear Acting on the Deck Panels per Unit Strip (1 ft)**

**1. Load Modifiers**

Load modifiers for deck design are shown in the flexure check.

**2. Strength I Limit State Load Factors**

[3.4.1]

Use the Strength I Limit State to determine the required shear resistance of the deck.

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results indicate that a single lane loaded with the design truck controls for shear.

**3. Strength I Limit State Shear Force**

Strength I Limit State factored shear force, one lane loaded = V<sub>u(m)</sub>

[Tables 3.4.1-1 and 3.4.1-2]

$$V_{u(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot [V_{truck(m)} + V_{lane(m)}]]$$

$$V_{u(m)} = 1.0 \cdot [1.25 \cdot (0.051) + 1.50 \cdot (0.239) + 1.75 \cdot 1.0 \cdot [3.555]] = 6.644 \text{ kips}$$

**Check Shear  
Resistance of Deck  
Panel**

**A. Factored Shear Resistance**

The factored shear force  $V_{u(m)}$  is the required shear resistance of the deck that needs to be compared with the actual factored shear resistance of the deck ( $V_r$ ).

[Eqns. 8.7-1, 8.7-2]

For a rectangular wood section  $V_r = \phi_v \cdot F_v \cdot b \cdot d_{lam} / 1.5$

[8.5.2.2]

**1. Resistance Factor**

Shear resistance factor =  $\phi_v = 0.75$

[8.4.4.2]

**2. Adjustment Factors for Reference Design Values**

Format conversion factor:  $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$

[8.4.4.3]

Wet Service factor =  $C_M = 0.875$

[8.4.4.9]

Time effect factor =  $C_\lambda = 0.80$

[Eqn. 8.4.4.1-2]

Adjusted design value =  $F_v = F_{vy0} \cdot C_{KF} \cdot C_M \cdot C_\lambda$   
 $F_v = 0.260 \cdot 3.33 \cdot 0.875 \cdot 0.80 = 0.606 \text{ ksi}$

**B. Deck Panel Shear Check**

Required deck shear resistance =  $V_{u(m)}$

For the deck to meet Strength I Limit State,  $V_{r(\text{prov})}$  must equal or exceed  $V_{u(m)}$ . As determined previously,  $V_{u(m)} = 6.644 \text{ kips}$ .

[Eqn. 8.7-2]

$$V_{r(\text{prov})} = \phi_v \frac{(F_v \cdot b \cdot d_{lam})}{1.5} = 0.75 \cdot \frac{(0.606 \cdot 12 \cdot 5.0)}{1.5} = 18.180 \text{ kips}$$

$$V_{u(m)} = 6.644 \text{ kips} \leq V_{r(\text{prov})} = 18.180 \text{ kips} \quad \text{OK}$$

**Check  
Compression  
Resistance**

Compression, or bearing of the deck on the beams, should be computed in accordance with the provisions of AASHTO LRFD for non-standard situations that provide a very narrow bearing area for the transverse deck. For this example, compression bearing on the glued laminated beams is not close to governing the design of the deck panel and so the calculation is not shown here. It usually will not govern a transverse deck design for a bridge of standard configuration. A bearing resistance calculation check for the longitudinal deck (on the pier caps) is shown in 8.7.1 Longitudinal Spike Laminated Timber Deck Design Example.

**Investigate  
Deflection  
Requirements  
[9.9.3.3]**

**A. Deck Live Load Deflection with Current Deck Parameters**

The final check for the transverse deck design to meet AASHTO LRFD is the deformation, or deflection, calculation. The design truck will have the most severe effect, and that is used for checking the transverse deck deflection.

**[3.6.1.3.3]**

As stated earlier, per AASHTO LRFD, when using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the design tandem (whichever results in the largest effect) shall be applied to the deck in determining live load force effects.

Deflections are to be calculated using Service I Limit State.

Calculate deck deflections for a transverse interconnected deck using a per foot width approach. This approach can be used on a glulam deck with shiplap joints and a spreader beam.

**[2.5.2.6.2]**

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

**[C2.5.2.6.2]**

As of note, if a plank deck or a non-interconnected panel deck is being analyzed, a different approach likely is required for the live load distribution, and an additional limitation of 0.10 inches relative deflection between adjacent edges is also required.

**1. Deck Stiffness**

Moment of inertia of one foot width of deck panels =  $I_{prov}$

$$I_{prov} = \frac{1}{12} \cdot b \cdot d_{lam}^3 = \frac{1}{12} \cdot 12 \cdot (5.0)^3 = 125.0 \text{ in}^4$$

Adjusted deck panel modulus of elasticity = E

**[8.4.4.3]**

Wet Service Factor for Modulus of Elasticity =  $C_M$

**[Table 8.4.4.3-2]**

$$C_M = 0.833$$

**[Eqn. 8.4.4.1-6]**

$$E = E_o \cdot C_M = 1700 \text{ ksi} \cdot 0.833 = 1416.1 \text{ ksi}$$

**2. Loads per Unit Strip Width (1 ft)**

Design truck load on deck span used for deflection calculations =  $P_{\Delta truck}$

Similar to calculations for the maximum positive bending moments, deflections are determined by considering the deck as a single simply-supported span between beams. Therefore, the point load on one deck span from design truck axle =  $P_{\text{truck}} = 16$  kips.

$P_{\text{truck}}$  expressed as per foot width =  $P_{\Delta\text{truck}}$ :

$$P_{\Delta\text{truck}} = P_{\text{truck}} \cdot 12 \text{ in} / E_s = P_{\text{truck}} \cdot 12 \text{ in} / 50 \text{ in}$$

$$P_{\Delta\text{truck}} = 16 \cdot 0.240 = 3.84 \text{ kips/ft}$$

**[Table 3.6.1.1.2-1]**

One lane loaded governs, the multiple presence factor =  $m = 1.20$

**[3.6.1.3]**

**[AISC 14<sup>th</sup> p. 3-215]**

### 3. Live Load Deflection Calculations

Deflection at deck midspan due to the design truck load =  $\Delta_{\text{truck}}$

$$\Delta_{\text{truck}} = \frac{m \cdot P_{\Delta\text{truck}} \cdot L_e^3}{48 \cdot E \cdot I_{\text{prov}}} = \frac{1.20 \cdot 3.84 \cdot (5.00 \cdot 12)^3}{48 \cdot 1416.1 \cdot 125} = 0.12 \text{ in}$$

The maximum deflection =  $\Delta_{\text{max}} = \Delta_{\text{truck}} = 0.12 \text{ in}$

**[2.5.2.6.2]**

Live load deflection limit at deck midspan =  $\Delta_{\text{max}}$

$$\Delta_{\text{max}} = L_e / 425 = 5.0 \text{ ft} \cdot 12 \text{ in} / 425 \cdot \text{ft} = 0.14 \text{ in}$$

$$\Delta = 0.12 \text{ in} \leq \Delta_{\text{max}} = 0.14 \text{ in}$$

OK

Deflections are also okay. Thus, the initial 5.0 inch deck panel depth and grade are adequate for the design.

### **8.8 Load Rating Examples**

#### **[References to MBE Section 6]**

This section demonstrates the calculation process for load rating wood bridge elements and contains several examples completed by the LRFR methodology. The Manual for Bridge Evaluation (MBE) published by AASHTO must be referenced as it governs bridge load ratings. All left hand references in this article are to the MBE.

The general load rating equation for determining the Rating Factor (RF) of a particular element, for the force effect being rated, is as follows:

$$\text{RF} = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

**[Eqn. 6A.4.2.1-1]**

#### **[6A.1.1]**

All existing, new, and rehabilitated bridges designed by LRFD must be load rated by the LRFR method. A structure properly designed and checked by the LRFD method should have the following minimum RF:  $RF_{Inv} = 1.0$ , and  $RF_{Oper} = 1.3$

#### **[6A.1.4]**

For cases in which the MBE is silent, the current AASHTO LRFD shall govern.

The following examples load rate the superstructure elements previously designed in the design examples (Section 8.7). Usually the force effects of moment and shear are checked for typical bridge superstructures. Bearing should also be checked if based on the engineer's judgment it could control the bridge load rating. In the following examples the force effects previously designed for, will be load rated.

#### **[Appendix A6A] [6A.1.5.1]**

Generally if the Design Load Rating, or first-level assessment, has an Inventory Rating Factor (RF) greater than or equal to 1.0, the bridge will not require posting. For simplicity of the following examples and to simply demonstrate the procedure, only the AASHTO LRFD HL-93 design vehicular live load will be load rated.

#### **[6A.2.2.1]**

The dead load effects on the structure shall be computed in accordance with the conditions existing at the time of the analysis. For a new bridge, the future wearing course used in design should not be included in the load rating calculation.

#### **[6A.2.3.2]**

One difference from design is traffic lane widths for live load application. In load ratings, roadway widths from 18 to 20 ft shall have two traffic lanes, each equal to one half the roadway width. Otherwise, live load placement is generally the same as for design.

**[6A.7.4.1]**

Requirements specific to wood structures are shown in 6A.7. For wood structures, rating factors for the design-load rating shall be based on the Strength I load combination.

**[6A.7.5]**

As with design, dynamic load allowance need not be applied to wood components.

**8.8.1 Longitudinal  
Spike Laminated  
Timber Deck Rating  
Example**

The variables in the general load rating equation need to be defined. Numbers from the design example for the longitudinal spike laminated timber deck will be used as applicable. The load rating will also be done on a per ft basis.

**Flexure Force effect**

**A. Capacity for Flexure Strength Limit State**

$$C = \phi_c \phi_s \phi R_n$$

**[6A.4.2.3]**

For a new bridge  $\phi_c = 1.00$

**[6A.4.2.4]**

For all timber bridges  $\phi_s = 1.00$

$$\text{For flexure, } \phi R_n = \phi_f M_n = \phi_f \cdot F_b \cdot S \cdot C_L$$

From Article 8.7.1 for this longitudinal spike laminated deck:

$$\phi_f = 0.85$$

$$F_b = 2.16 \text{ ksi for Douglas Fir-Larch Deck (No. 1)}$$

$$S = \frac{12 \text{ in} \cdot d^2}{6} = \frac{12 \text{ in} \cdot 14^2}{6} = 392 \cdot \text{in}^3$$

$$C_L = 1.0$$

$$\phi_f M_n = 0.85 \cdot 2.16 \cdot 392 \cdot 1.0 = 719.71 \text{ kip} \cdot \text{in}$$

$$\text{Therefore, } C = 1.00 \cdot 1.00 \cdot 719.71 = 719.71 \text{ kip} \cdot \text{in}$$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the live load factor:

$$\gamma_{LL} = 1.35$$

**C. Force Effects for Flexure**

The force effects for flexure (bending moments) were calculated in Article 8.7.1. The values shown here are taken from Table 8.7.1.1 (except that the FWC is removed from  $M_{dw}$ ):

$$M_{dc} = 3.82 \text{ kip} \cdot \text{ft} = 45.84 \text{ kip} \cdot \text{in}$$

$$M_{dw} = 2.84 \text{ kip} \cdot \text{ft} = 34.08 \text{ kip} \cdot \text{in}$$

$$M_{\text{tandem}} = 21.40 \text{ kip} \cdot \text{ft} = 256.80 \text{ kip} \cdot \text{in} \text{ (for two lanes loaded, tandem governs over truck)}$$

$$M_{\text{lane}} = 3.56 \text{ kip} \cdot \text{ft} = 42.72 \text{ kip} \cdot \text{in} \text{ (for two lanes loaded)}$$

**Rating Factors****A. Calculate Inventory Rating Factor for Flexure**

$$RF_{\text{Inv}} = \frac{719.71 - (1.25)(45.84) - (1.50)(34.08)}{(1.75)(256.80 + 42.72)}$$

$$RF_{\text{Inv}} = 1.17$$

**B. Calculate Operating Rating Factor for Flexure**

$$RF_{\text{Oper}} = \frac{719.71 - (1.25)(45.84) - (1.50)(34.08)}{(1.35)(256.80 + 42.72)}$$

$$RF_{\text{Oper}} = 1.51$$

**8.8.2 Glulam Beam  
Superstructure  
Rating Example**

Similar to the example above, the variables in the general load rating equation need to be defined for the element (in this case beam) and force effect being rated. Numbers from the design example for the glulam beam superstructure will be used as applicable. The load rating will be done for an interior beam because that was previously shown to govern.

**Flexure Force effect  
[Eqn. 6A.4.2.1-2]**

**A. Capacity for Flexure Strength Limit State**

$$C = \phi_c \phi_s \phi R_n$$

**[6A.4.2.3]**

For a new bridge  $\phi_c = 1.00$

**[6A.4.2.4]**

For all timber bridges  $\phi_s = 1.00$

For flexure,  $\phi R_n = \phi_f M_n = \phi_f \cdot F_b \cdot S \cdot C_L$

Article 8.7.3 for the glulam beam in flexure:

$$\phi_f = 0.85$$

$F_b = 3.97$  ksi for SP/SP glulam beam (24F-V3)

$$S_{\text{prov}} = \frac{b \cdot d^2}{6} = \frac{8.5 \cdot 46.75^2}{6} = 3096.21 \text{ in}^3$$

$$C_L = 1.0$$

$$\phi_f M_n = 0.85 \cdot 3.97 \cdot 3096.21 \cdot 1.0 = 10,448.16 \text{ kip} \cdot \text{in}$$

Therefore,  $C = 1.00 \cdot 1.00 \cdot 10,448.16 = 10,448.16 \text{ kip} \cdot \text{in}$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the Live Load Factor:

$$\gamma_{LL} = 1.35$$

**C. Force Effects for Flexure**

The force effects for flexure (bending moments) were calculated in Article 8.7.3. The values shown here are taken from Table 8.7.3.1 (except that the FWC is removed from  $M_{dw}$ ):

$$M_{dc} = 69.95 \text{ kip} \cdot \text{ft} = 839.40 \text{ kip} \cdot \text{in}$$

$$M_{dw} = 61.30 \text{ kip} \cdot \text{ft} = 735.60 \text{ kip} \cdot \text{in}$$

$$M_{\text{truck}} = 291.12 \text{ kip} \cdot \text{ft} = 3493.44 \text{ kip} \cdot \text{in} \text{ (truck governs over tandem)}$$

$$M_{\text{lane}} = 84.66 \text{ kip} \cdot \text{ft} = 1015.92 \text{ kip} \cdot \text{in}$$

**Rating Factors****A. Calculate Inventory Rating Factor for Flexure**

$$RF_{\text{Inv}} = \frac{10,448.16 - (1.25)(839.40) - (1.50)(735.60)}{(1.75)(3493.44 + 1015.92)}$$

$$RF_{\text{Inv}} = 1.05$$

**B. Calculate Operating Rating Factor for Flexure**

$$RF_{\text{Oper}} = \frac{10,448.16 - (1.25)(839.40) - (1.50)(735.60)}{(1.35)(3493.44 + 1015.92)}$$

$$RF_{\text{Oper}} = 1.36$$

**Shear Force effect**

[Eqn. 6A.4.2.1-2]

**A. Capacity for Shear Strength Limit State**

$$C = \phi_c \phi_s \phi_{R_n}$$

[6A.4.2.3]

For a new bridge  $\phi_c = 1.00$

[6A.4.2.4]

For all timber bridges  $\phi_s = 1.00$

$$\text{For shear, } \phi_{R_n} = \phi_v V_n = \phi_v \cdot F_v \cdot w_{bm} \cdot d_{bm} / 1.5$$

From Article 8.7.3 for the glulam beam in shear:

$$\phi_v = 0.75$$

$$F_v = 0.699 \text{ ksi for SP/SP glulam beam (24F-V3)}$$

$$d_{bm} = 46.75 \text{ in}$$

$$w_{bm} = 8.5 \text{ in}$$

$$\phi_v V_n = 0.75 \cdot 0.699 \cdot 8.5 \cdot 46.75 / 1.5 = 138.88 \text{ kips}$$

$$\text{Therefore, } C = 1.00 \cdot 1.00 \cdot 138.88 = 138.88 \text{ kips}$$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the live load factor:

$$\gamma_{LL} = 1.35$$

**C. Force Effects for Shear**

The force effects for shear were calculated in Article 8.7.3. The values shown here are taken from that example at a distance “ $d_{beam}$ ” away from the support (the FWC is not included in  $V_{dw}$ ):

$$V_{dc} = 5.18 \text{ kips}$$

$$V_{dw} = 4.67 \text{ kips}$$

$$V_{truck} = 38.00 \text{ kips (truck governs over tandem)}$$

$$V_{lane} = 6.72 \text{ kips}$$

$$V_{LL} = 26.83 \text{ kips (this is the distributed LL per beam)}$$

**Rating Factors****A. Calculate Inventory Rating Factor for Shear**

$$RF_{Inv} = \frac{138.88 - (1.25)(5.18) - (1.50)(4.67)}{(1.75)(26.83)}$$

$$RF_{Inv} = 2.67$$

**B. Calculate Operating Rating Factor for Shear**

$$RF_{Oper} = \frac{138.88 - (1.25)(5.18) - (1.50)(4.67)}{(1.35)(26.83)}$$

$$RF_{Oper} = 3.46$$

**Compressive Force effect**

**[Eqn. 6A.4.2.1-2]**

**[6A.4.2.3]**

**A. Capacity for Compressive Strength Limit State**

$$C = \phi_c \phi_s \phi_{Rn}$$

For a new bridge  $\phi_c = 1.00$

**[6A.4.2.4]**

For all timber bridges  $\phi_s = 1.00$

For compression,  $\phi R_n = \phi_{cperp} P_n = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b$

From Article 8.7.3 for this glulam beam:

$$\phi_{cperp} = 0.90$$

$$F_{cp} = 0.731 \text{ ksi for SP/SP glulam beam (24F-V3)}$$

$$\text{Bearing Area} = A_b = L_b \times W_{bm} = 18.0 \times 8.5 = 153.0 \text{ in}^2$$

$$C_b = 1.00$$

$$\phi_{cperp} P_n = 0.90 \cdot 0.731 \cdot 153.0 \cdot 1.0 = 100.66 \text{ kips}$$

$$\text{Therefore, } C = 1.00 \cdot 1.00 \cdot 100.66 = 100.66 \text{ kips}$$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the Live Load Factor:

$$\gamma_{LL} = 1.35$$

**C. Force Effects for Compression**

The force effects for compression were calculated in Article 8.7.3. The values shown here are taken from that example (the FWC is not included):

$$R_{dc} = 6.84 \text{ kips}$$

$$R_{dw} = 5.84 \text{ kips}$$

$$R_{truck} = 56.00 \text{ kips (truck governs over tandem)}$$

$$R_{lane} = 13.40 \text{ kips}$$

$$R_{LL} = 41.64 \text{ kips (this is the distributed LL per beam)}$$

**Rating Factors****A. Calculate Inventory Rating Factor for Compression**

$$RF_{Inv} = \frac{100.66 - (1.25)(6.84) - (1.50)(5.84)}{(1.75)(41.64)}$$

$$RF_{Inv} = 1.14$$

**B. Calculate Operating Rating Factor for Compression**

$$RF_{Oper} = \frac{100.66 - (1.25)(6.84) - (1.50)(5.84)}{(1.35)(41.64)}$$

$$RF_{Oper} = 1.48$$

**[This page intentionally left blank.]**

**8.8.3 Transverse  
Deck Rating  
Examples**

The variables in the general load rating equation need to be defined for the transverse decks and force effect being rated. Numbers from the design example for the transverse decks will be used as applicable. The load rating will also be done on a per ft basis.

**Transverse Spike  
Laminated Deck**

The transverse spike laminated deck will be load rated first, for the flexure and the shear force effects.

**Flexure Force  
effect  
[Eqn. 6A.4.2.1-2]**

**A. Capacity for Flexure Strength Limit State**

$$C = \phi_c \phi_s \phi R_n$$

**[6A.4.2.3]**

For a new bridge  $\phi_c = 1.00$

**[6A.4.2.4]**

For all timber bridges  $\phi_s = 1.00$

For flexure,  $\phi R_n = \phi_f M_n = \phi_f \cdot F_b \cdot S \cdot C_L$

From Article 8.7.4 for this transverse spike laminated deck in flexure:

$$\phi_f = 0.85$$

$F_b = 2.152$  ksi for Douglas Fir-Larch Deck (No. 2)

$$S = \frac{b \cdot d^2}{6} = \frac{12 \text{ in} \cdot 5.75^2}{6} = 66.13 \cdot \text{in}^3$$

$$C_L = 1.0$$

$$\phi_f M_n = 0.85 \cdot 2.152 \cdot 66.13 \cdot 1.0 = 120.97 \text{ kip} \cdot \text{in}$$

Therefore,  $C = 1.00 \cdot 1.00 \cdot 120.97 = 120.97 \text{ kip} \cdot \text{in}$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the Live Load Factor:

$$\gamma_{LL} = 1.35$$

**C. Force Effects for Flexure**

The force effects for flexure (bending moments) were calculated in Article 8.7.4 on a per ft basis. The values shown here are taken from Table 8.7.4.1 (except that the FWC is removed from  $M_{dw}$ ):

$$M_{dc} = 0.089 \text{ kip} \cdot \text{ft} = 1.07 \text{ kip} \cdot \text{in}$$

$$M_{dw} = 0.269 \text{ kip} \cdot \text{ft} = 3.23 \text{ kip} \cdot \text{in}$$

$$M_{truck} = 4.572 \text{ kip} \cdot \text{ft} = 54.86 \text{ kip} \cdot \text{in} \text{ (truck governs over tandem)}$$

**Rating Factors****A. Calculate Inventory Rating Factor for Flexure**

$$RF_{Inv} = \frac{120.97 - (1.25)(1.07) - (1.50)(3.23)}{(1.75)(54.86)}$$

$$RF_{Inv} = 1.20$$

**B. Calculate Operating Rating Factor for Flexure**

$$RF_{Oper} = \frac{120.97 - (1.25)(1.07) - (1.50)(3.23)}{(1.35)(54.86)}$$

$$RF_{Oper} = 1.55$$

**Shear Force effect**  
**[Eqn. 6A.4.2.1-2]****A. Capacity for Shear Strength Limit State**

$$C = \phi_c \phi_s \phi R_n$$

**[6A.4.2.3]**

For a new bridge  $\phi_c = 1.00$

**[6A.4.2.4]**

For all timber bridges  $\phi_s = 1.00$

For shear,  $\phi R_n = \phi_v V_n = \phi_v \cdot F_v \cdot b \cdot d_{lam} / 1.5$

From Article 8.7.4 for this transverse spike laminated deck in shear:

$$\phi_v = 0.75$$

$$F_v = 0.372 \text{ ksi for Douglas Fir-Larch Deck (No. 2)}$$

$$b = 12.0 \text{ in}$$

$$d_{lam} = 5.75 \text{ in}$$

$$\phi_v V_n = 0.75 \cdot 0.372 \cdot 12.0 \cdot 5.75 / 1.5 = 12.83 \text{ kips}$$

$$\text{Therefore, } C = 1.00 \cdot 1.00 \cdot 12.83 = 12.83 \text{ kips}$$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the Live Load Factor:

$$\gamma_{LL} = 1.35$$

**C. Force Effects for Shear**

The force effects for shear were calculated in Example 8.7.4 on a per ft basis. The values shown here are taken at a distance "d<sub>lam</sub>" away from the support (the FWC is not included in V<sub>dw</sub>):

$$V_{dc} = 0.059 \text{ kips}$$

$$V_{dw} = 0.190 \text{ kips}$$

$$V_{truck} = 2.775 \text{ kips (truck governs over tandem)}$$

**Rating Factors****A. Calculate Inventory Rating Factor for Shear**

$$RF_{Inv} = \frac{12.83 - (1.25)(0.059) - (1.50)(0.190)}{(1.75)(2.775)}$$

$$RF_{Inv} = 2.57$$

**B. Calculate Operating Rating Factor for Shear**

$$RF_{Oper} = \frac{12.83 - (1.25)(0.059) - (1.50)(0.190)}{(1.35)(2.775)}$$

$$RF_{Oper} = 3.33$$

**Transverse Glued Laminated Deck**

The transverse glued laminated deck will be load rated next, for the flexure and the shear force effects.

**Flexure Force effect**

**[Eqn. 6A.4.2.1-2]**

**A. Capacity for Flexure Strength Limit State**

$$C = \phi_c \phi_s \phi R_n$$

**[6A.4.2.3]**

For a new bridge  $\phi_c = 1.00$

**[6A.4.2.4]**

For all timber bridges  $\phi_s = 1.00$

For flexure,  $\phi R_n = \phi_f M_n = \phi_f \cdot F_b \cdot S \cdot C_L$

From Article 8.7.4 for this transverse glued laminated deck in flexure:

$$\phi_f = 0.85$$

$F_b = 4.140$  ksi for Southern Pine (ID No. 48)

$$S = \frac{12 \text{ in} \cdot d^2}{6} = \frac{12 \text{ in} \cdot 5.0^2}{6} = 50.0 \cdot \text{in}^3$$

$$C_L = 1.0$$

$$\phi_f M_n = 0.85 \cdot 4.14 \cdot 50.0 \cdot 1.0 = 175.95 \text{ kip} \cdot \text{in}$$

Therefore,  $C = 1.00 \cdot 1.00 \cdot 175.95 = 175.95 \text{ kip} \cdot \text{in}$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the Live Load Factor:

$$\gamma_{LL} = 1.35$$

**C. Force Effects for Flexure**

The force effects for flexure (bending moments) were calculated in Article 8.7.4 on a per ft basis. The values shown here are taken from Table 8.7.4.2 (except that the FWC is removed from  $M_{dw}$ ):

$$M_{dc} = 0.077 \text{ kip} \cdot \text{ft} = 0.92 \text{ kip} \cdot \text{in}$$

$$M_{dw} = 0.269 \text{ kip} \cdot \text{ft} = 3.23 \text{ kip} \cdot \text{in}$$

$$M_{truck} = 5.76 \text{ kip} \cdot \text{ft} = 69.12 \text{ kip} \cdot \text{in} \text{ (truck governs over tandem)}$$

**Rating Factors****A. Calculate Inventory Rating Factor for Flexure**

$$RF_{Inv} = \frac{175.95 - (1.25)(0.92) - (1.50)(3.23)}{(1.75)(69.12)}$$

$$RF_{Inv} = 1.41$$

**B. Calculate Operating Rating Factor for Flexure**

$$RF_{Oper} = \frac{175.95 - (1.25)(0.92) - (1.50)(3.23)}{(1.35)(69.12)}$$

$$RF_{Oper} = 1.82$$

**Shear Force effect**  
**[Eqn. 6A.4.2.1-2]****A. Capacity for Shear Strength Limit State**

$$C = \phi_c \phi_s \phi R_n$$

**[6A.4.2.3]**

For a new bridge  $\phi_c = 1.00$

**[6A.4.2.4]**

For all timber bridges  $\phi_s = 1.00$

For shear,  $\phi R_n = \phi_v V_n = \phi_v \cdot F_v \cdot b \cdot d_{lam} / 1.5$

From Article 8.7.4 for this transverse glued laminated deck in shear:

$$\phi_v = 0.75$$

$$F_v = 0.606 \text{ ksi for Southern Pine (ID No. 48)}$$

$$b = 12.0 \text{ in}$$

$$d_{lam} = 5.0 \text{ in}$$

$$\phi_v V_n = 0.75 \cdot 0.606 \cdot 12.0 \cdot 5.0 / 1.5 = 18.18 \text{ kips}$$

$$\text{Therefore, } C = 1.00 \cdot 1.00 \cdot 18.18 = 18.18 \text{ kips}$$

**B. Load Factors**

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

**[Table 6A.4.2.2-1]**

$$\gamma_{DC} = 1.25$$

$$\gamma_{DW} = 1.50$$

$\gamma_P = 1.0$  (there are no other permanent loads and so this will be neglected in the final calculation)

$$\gamma_{LL} = 1.75$$

The only change to the Operating Rating level is for the live load factor:

$$\gamma_{LL} = 1.35$$

### C. Force Effects for Shear

The force effects for shear were calculated in Article 8.7.4 on a per ft basis. The values shown here are taken at a distance "d<sub>lam</sub>" away from the support (the FWC is not included in V<sub>dw</sub>):

$$V_{dc} = 0.051 \text{ kips}$$

$$V_{dw} = 0.190 \text{ kips}$$

$$V_{truck} = 3.555 \text{ kips (truck governs over tandem)}$$

## Rating Factors

### A. Calculate Inventory Rating Factor for Shear

$$RF_{Inv} = \frac{18.18 - (1.25)(0.051) - (1.50)(0.190)}{(1.75)(3.555)}$$

$$RF_{Inv} = 2.87$$

### B. Calculate Operating Rating Factor for Shear

$$RF_{Oper} = \frac{18.18 - (1.25)(0.051) - (1.50)(0.190)}{(1.35)(3.555)}$$

$$RF_{Oper} = 3.72$$

**9. DECKS AND DECK SYSTEMS**

Reinforced concrete decks on girders are the predominant type of deck used on highway bridges in Minnesota. The deck is the structural element that transfers vehicle and pedestrian loads to the girders. It is analyzed as a continuous beam with the girders acting as supports. The top and bottom primary moment resisting reinforcement runs transversely in the deck. The concrete stool between the girder top flange and the deck bottom varies to allow placement of the deck to the proper elevation.

Only reinforced concrete decks supported on girders are covered in this section. Practices for slab type bridges, where the superstructure does not contain girder supports, are located in Article 5.3 of this manual.

Timber decks may be used on secondary roads and temporary bridges as part of the superstructure. Guidance for the design of timber decks is provided in Section 8.

Specialized deck systems are used for railroad bridges. A common design is a thru-girder system with floor beams supporting a bent plate. This channel shaped bent plate holds the ballast on which the rails are supported. These specialized deck systems are not currently covered in this manual.

**9.1 General**

**Bridge Deck Protection Policy**

Refer to BDM Article 2.4.1.1.2 for the bridge deck protection policy.

**9.1.1 Deck Drainage**

**Deck Drainage Considerations**

The design of a deck requires:

- Removing water from the driving surface using a crown cross-section to protect against potential hydroplaning.
- Channeling drainage water away from the bridge and features below the bridge using road grades and end slopes respectively.

Deck drains and drainage systems on bridges are strongly discouraged due to their high maintenance requirements. Debris tends to build up in the drains, causing plugging of the system. Drainage systems are also prone to leakage, which is especially a problem for box type structures where the system runs inside the box. Bridges with lengths less than 500 feet that are located over lakes or streams can usually be designed such that deck drains are not necessary. Bridges that are longer than 500 feet may have problems with deck flooding in severe rainstorms, and may require deck drains. The Bridge Waterways Unit will work with the Bridge Preliminary

Plans Unit to investigate the need for deck drains and include the requirements, if any, in the Preliminary Bridge Plan.

### **Superstructure Drains**

When drainage systems are required on bridges, avoid direct runoff into "waters of the state", as defined in *Mn. Statute 115.01, Subdivision 22*.

Extend drains a minimum of 1 inch below the bottom of superstructure. Use a longer extension (up to 1 foot maximum), if possible, where geometry below allows. See Standard Bridge Detail B701, B702, B705, or B706.

Avoid drain outlets over roadways, shoulders, sidewalks/trails, streams, railroad tracks, and end slopes. Drains placed over riprap require the area to be grouted, or a grouted flume section provided. At down spouts or deck drains, provide splash blocks.

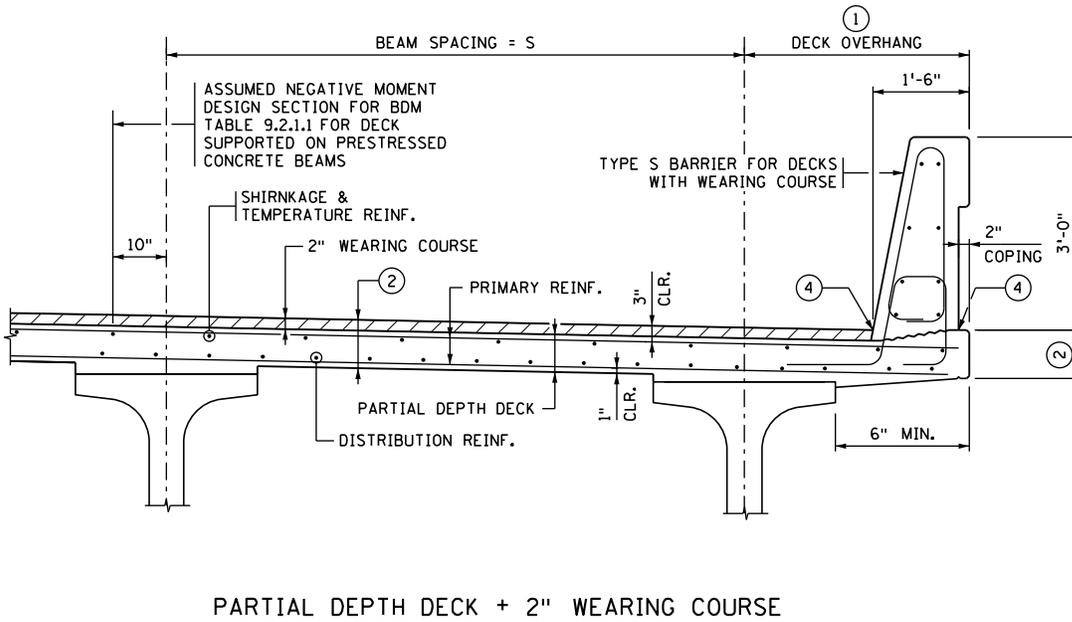
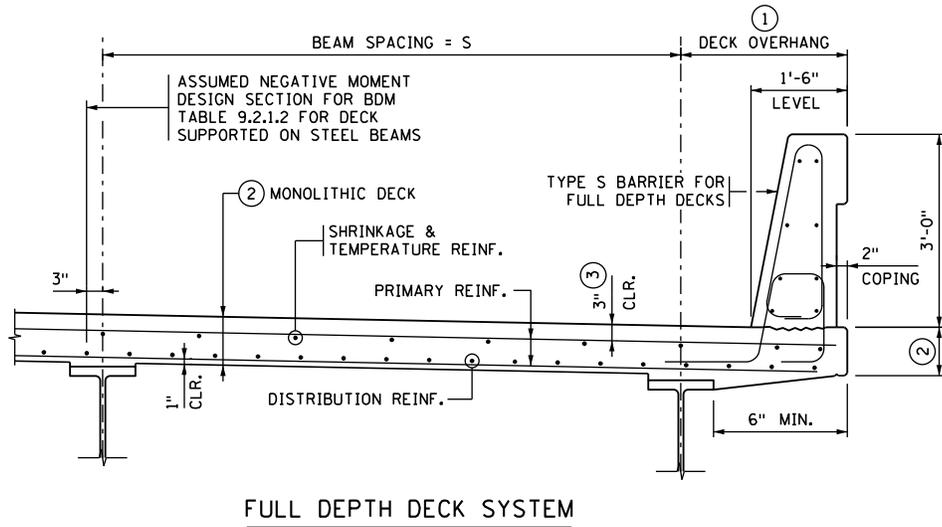
Avoid drainage details that include flat elements (grades less than 5%). Pipes and drainage elements with flat profiles tend to collect debris and plug.

Note that special drainage requirements are necessary for bridges where a Corps of Engineers "404 permit" is required. The Bridge Waterways Unit may also require the addition of containment and treatment features to the project for bridges located in or near scenic waterways or near public water supply sources.

Provide the materials and gages for corrugated metal (C.M.) drains and semi-circle deck drains, such as those used on railroad bridges, in the plan details.

## **9.2 Concrete Deck on Beams**

Figure 9.2.1 illustrates the two most common concrete deck systems used. See the deck protection policy in BDM Article 2.4.1.1.2 for determination of which deck system to use for a given bridge project. The upper portion of the figure shows a monolithic deck constructed with a single concrete pour. The lower portion illustrates a deck with a wearing course.



**GEOMETRY AND DESIGN ASSUMPTIONS FOR MOST COMMON CONCRETE DECK SYSTEMS**

**NOTES:**

EITHER DECK SYSTEM CAN BE USED WITH EITHER BEAM TYPE.  
 REINFORCEMENT CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER MAY DIFFER FOR STAINLESS OR GFRP BARS.

- ① PER BDM ARTICLE 2.4.1.1.1 GENERALLY LIMIT THE OVERHANG TO THE SMALLEST OF:
  - BEAM DEPTH
  - 40% OF BEAM SPACING
  - DECK COPING WIDTH + BARRIER WIDTH + 1'-0" + 1/2 FLANGE WIDTH
- ② FOR VEHICULAR BRIDGES, REQ'D. DECK THICKNESS AND EDGE OF DECK THICKNESS IS DEPENDENT ON BEAM SPACING (SEE TABLES 9.2.1.1 & 9.2.1.2). FOR PEDESTRIAN BRIDGES, USE 7" DECK THICKNESS WITH NO WEARING COURSE.
- ③ FOR PEDESTRIAN BRIDGES, PROVIDE 2" CLEAR COVER.
- ④ GUTTER LINE AND COPING ELEVATION TO MATCH.

**Figure 9.2.1**

**9.2.1 Deck Design  
and Detailing****Design**

The default reinforcement bar type used in concrete bridge decks is an epoxy coated bar with a yield strength,  $F_y$ , equal to 60 ksi which meets the material requirements of ASTM A615.

In special cases, as outlined in Technical Memorandum No. 17-02-B-01, use stainless steel reinforcement and design accordingly. Note that stainless steel bars have a higher yield strength and different clear cover requirements.

Use of glass fiber reinforced polymer (GFRP) reinforcement bars has been limited to a small number of specific projects and is not to be specified unless approved by the State Bridge Design Engineer.

For design of conventionally reinforced concrete decks, the following requirements apply:

- Use the traditional approximate method of analysis for design of the top and bottom transverse reinforcement. Do not use the empirical deck design method in LRFD Article 9.7.2.
- For analysis, assume the deck is a continuous transverse strip with the beams below as supports.
- For skews less than or equal to  $20^\circ$ , detail deck transverse bars parallel to the skew. For design of the transverse bars, use the beam spacing measured along the skew for the deck span length.
- For skews greater than  $20^\circ$ , detail deck transverse bars at right angles to the centerline of roadway. For design of the transverse bars, use the beam spacing measured normal to the roadway centerline for the deck span length.
- For dead load, include deck self-weight plus a future wearing course of 20 psf. Apply a load factor of 1.25.
- If LRFD Appendix A4 assumptions and limitations are met, use the live load moments provided in LRFD Table A4-1 for design. Apply negative moment live load at the design section specified in LRFD Article 4.6.2.1.6.
- For decks without a wearing course, assume  $\frac{1}{2}$  inch of wear when determining structural depth,  $d$ , for the bottom transverse reinforcement. For decks with a wearing course, do not include the wearing course (sacrificial) when determining structural depth,  $d$ , for the bottom transverse reinforcement.
- Check crack control per LRFD Article 5.6.7 using the Class 2 exposure condition ( $\gamma_e = 0.75$ ). In addition, although the actual concrete clear cover to top transverse bars may exceed 2 inches, calculate  $d_c$  using a maximum clear concrete cover equal to 2 inches.

- For bottom longitudinal reinforcement, provide distribution reinforcement per LRFD Article 9.7.3.2. For bridges with varying beam spacing, base the distribution reinforcement for each unit (where a unit is defined as the number of spans between expansion joints) on the widest beam spacing found within the unit.
  - For the deck region in non-pier areas, provide top longitudinal reinforcement that meets the requirements for shrinkage and temperature reinforcement in LRFD Article 5.10.6.
  - For the deck region over/near a pier, provide top longitudinal reinforcement consistent with the superstructure modeling assumptions:
    - Where deck is continuous, but beams are not continuous, provide reinforcement per Figure 9.2.1.8.
    - Where deck and prestressed beams are continuous, design reinforcement for factored negative moment.
    - Where deck and steel beams are continuous, design reinforcement for factored negative moment and meet requirements of LRFD Article 6.10.1.7. See Figure 9.2.1.9 for additional information.
  - Design the deck overhang to carry the lesser of:
    - the  $M_c$  corresponding to the rail capacity  $R_w$
    - $\frac{4}{3} \cdot M_{cFt}$ ,  
where  $M_{cFt}$  = the barrier flexural resistance about the longitudinal axis corresponding to  $F_t$ .
- See discussion below and Memo to Designers (2017-01) for additional guidance on deck overhang design.

Tables 9.2.1.1 and 9.2.1.2 provide minimum reinforcement requirements based on the traditional deck design method for decks supported on precast pretensioned concrete beams and steel beams, respectively. The tables may be used for all LRFD deck designs that fit the assumptions, as well as for decks of bridges originally designed by the AASHTO Standard Specifications Load Factor method (bridge widenings).

#### **[5.10.6]**

The transverse reinforcement given in Tables 9.2.1.1 and 9.2.1.2 is adequate for interior region deck overhangs (measured from centerline of beam to edge of deck) of up to 40% of the beam spacing when a 36 inch tall Type S concrete barrier which meets NCHRP Report 350 Test Level 4 (TL-4) is used. See Standard Details Manual Part II Figures 5-397.138(A) through 5-397.139(D) for Type S concrete barrier details. For exterior region overhangs (regions where the longitudinal barrier reinforcement is discontinuous, such as end of bridge joints and expansion joints), refer to Memo to Designers (2017-01) for overhang reinforcement requirements.

Decks with geometry or loads that fall outside the Table 9.2.1.1 and 9.2.1.2 assumptions require a special design.

Overhangs are to be designed to meet the strength requirements of Section 13. LRFD A13.4.2 specifies that the moment resistance of the deck overhang must be greater than or equal to the barrier flexural resistance about the longitudinal axis,  $M_c$ . This ensures that the deck will be strong enough to force the yield line failure mechanism to occur in the barrier. However, the barrier flexural resistance in the interior regions can be substantially larger than that required to resist the design collision force. For example, the interior panel of a TL-4, 36 inch Type S barrier on a deck with a wearing course has a capacity  $R_w = 117.4$  kips and a corresponding  $M_c = 17.1$  kip-ft/ft, which is well above the design collision force  $F_t = 54$  kips and corresponding  $M_{cFt} = 7.9$  kip-ft/ft for an NCHRP Report 350 Test Level 4 barrier. Because of the large difference between barrier capacity and collision force, MnDOT requires the deck overhang to carry the lesser of:

- the  $M_c$  corresponding to the rail capacity  $R_w$
- $\frac{4}{3} \cdot M_{cFt}$   
where  $M_{cFt} =$  the barrier flexural resistance about the longitudinal axis corresponding to  $F_t$

Because the yield line equations in LRFD assume the collision load is applied at the top of the barrier, adjustment of collision force  $F_t$  for the difference between the barrier height and height of application is required when  $\frac{4}{3} \cdot M_{cFt}$  governs. Refer to the design example in BDM Article 9.3 and the Memo to Designers (2017-01) for more information.

### Geometry

Figures 9.2.1.4 through 9.2.1.7 show standard practice deck details. Typical deck reinforcement layouts at deck edges and medians are illustrated in the figures.

Use a uniform deck thickness for all spans based on the minimum thickness required for the widest beam spacing. For new bridges, use a 9 inch minimum deck thickness on all vehicular structures and a 7 inch minimum deck thickness on pedestrian bridges. For bridge repair projects on vehicular bridges, a lesser deck thickness (8 inch minimum) may be used when approved by the Regional Bridge Construction Engineer to achieve an acceptable load rating.

For the edge-of-deck thickness, use a uniform thickness in all spans. Refer to Memo to Designers (2017-01) for additional guidance regarding the edge-of-deck thickness.

The standard height for bridge sidewalks at the gutter line is 6 inches above the top of roadway. For bridge medians, match approach roadway median shape and height as shown in the preliminary bridge plan.

Dimension the bottom of deck on the outside of the fascia beam at 1 inch below the top of the beam for prestressed concrete beams. For steel beams, detail the bottom of deck on the outside of the fascia beam to meet the bottom of the top flange. See Figures 9.2.1.4 through 9.2.1.7.

Check the slope of the bottom of the deck on overhangs. Confirm that the bottom edge of the deck is higher than the location next to the beam top flange.

### **Detailing**

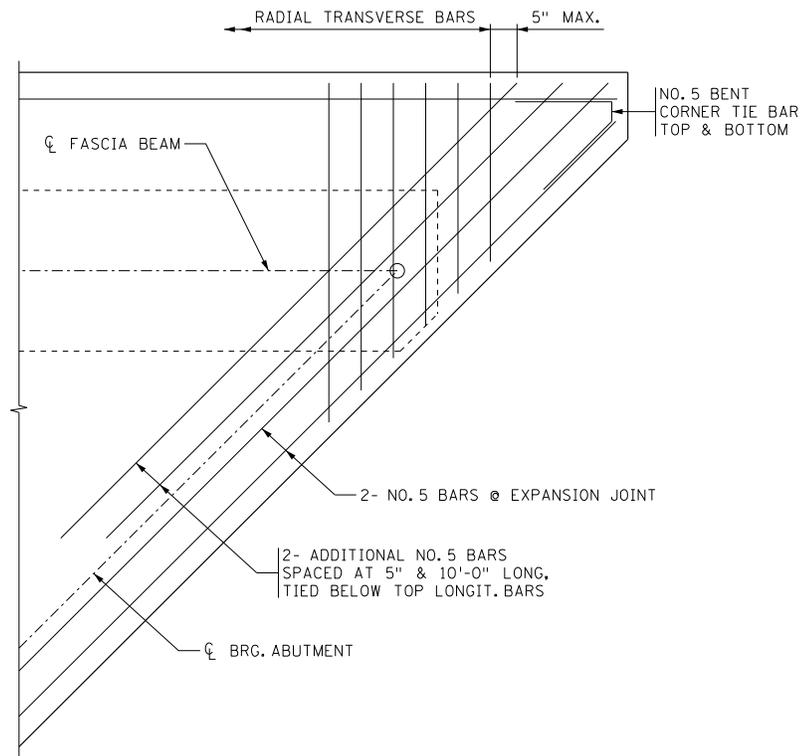
For main transverse deck reinforcement, provide straight bars located in both the top and the bottom reinforcing mats. Refer to Memo to Designers (2017-01) for some exceptions to this in the deck overhang where hooked bars are required.

The main transverse reinforcement will vary with the beam spacing. For skewed bridges where the beam spacing changes from one span to another, continue the reinforcement for the wider beam spacing until the reinforcement is completely outside of the span with the wider beam spacing.

For the acute corners of highly skewed bridges, detail the deck reinforcement as follows: In addition to the 2-#5 bars that run parallel to the expansion joint at the end of the deck, place 2 top mat #5 bars that are 10 feet long and run parallel to the joint with a spacing of 5 inches. Also, run a series of radial transverse bars that shorten as they progress into the corner. Finally, place a bent bar in the corner that ties to the outside deck longitudinal bar and the end bar running parallel to the joint. See Figure 9.2.1.1.

Add a longitudinal tie at the end of the deck if the deck projects past the end of the diaphragm more than 1 foot.

For bridges with transverse deck reinforcement parallel to the skew, dimension transverse bar spacing along edge of deck.



TYPICAL DECK REINFORCEMENT PLAN FOR HIGHLY SKEWED CORNERS

**Figure 9.2.1.1**

Several detailing practices are to be used near piers:

- Detail longitudinal steel (temperature and distribution) as continuous over piers.
- Provide additional longitudinal steel to minimize transverse deck cracking. See Figures 9.2.1.8 and 9.2.1.9.
- For decks supported on non-continuous prestressed beams, detail a partial depth sawcut in the deck over the pier backfilled with a sealant. See Figure 9.2.1.10.
- Place polystyrene on the corners of prestressed concrete beam bridges with skews greater than 20° to reduce wandering of the transverse deck crack at the centerline of pier. See Figure 9.2.1.10.

**Deck Placement Sequence**

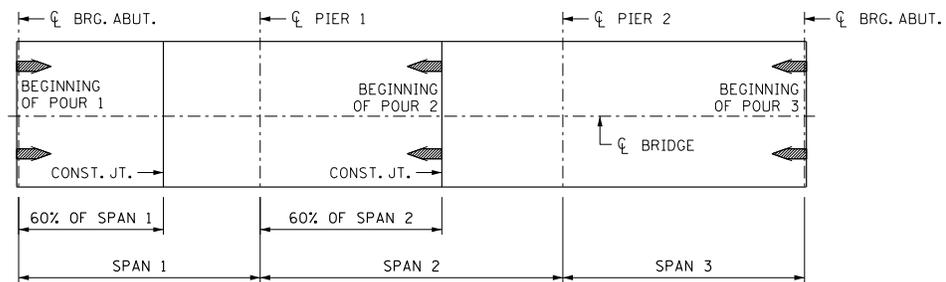
One contributor to through-deck transverse cracking is inadequate sequencing of deck pours. Provide a deck placement sequence for the following types of bridges:

- Bridges with decks wider than 90 feet.
- Continuous bridges with spans exceeding 150 feet.

- Bridges where the concrete placement rate is lower than 60% of the span length per hour. (Note that a single pump truck can be assumed to maintain a pour rate of 70 cubic yards per hour.)

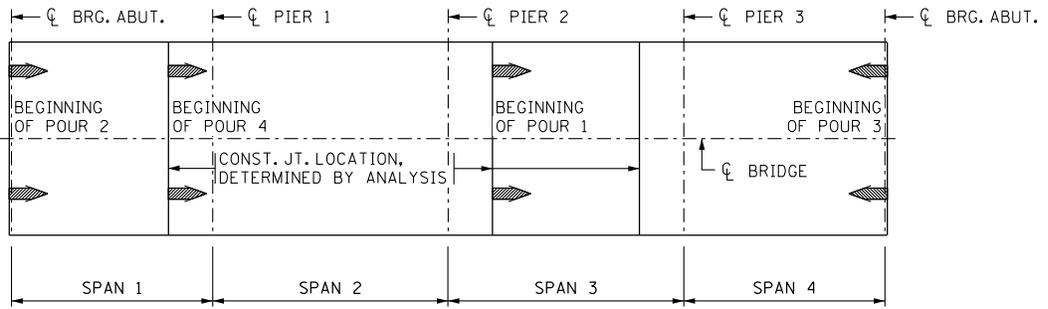
The overall goal of the deck placement sequence is have minimal tension in the deck due to self-weight when the entire deck is complete.

Generally, for continuous superstructures containing span lengths between 150 and 200 feet, locate the transverse construction joint for the first pour at the 0.6 point of the first span. Start the following pour at the 0.6 point of the adjacent span and proceed toward and terminate at the end of the previous pour. Continue this pattern for all interior spans. The last placement will extend from the end of the bridge to the previous placement. A typical deck placement sequence for a 3 span bridge fitting the above criteria is shown in Figure 9.2.1.2.



**Figure 9.2.1.2**

For continuous superstructures containing span lengths greater than 200 feet, conduct an analysis to determine construction joint locations for the deck pour sequence. For the initial trial, set the joint locations at points of dead load contraflexure. Choose a pour sequence that minimizes upward deflections in previously placed spans (i.e. longer pour sections should be placed before shorter adjacent sections). Place positive moment sections prior to negative moment sections. Next, analyze the deck for the initial trial pour sequence to determine the cumulative stresses in the deck. Then begin adjusting construction joint locations and reanalyzing until the pour sequence with the lowest tension stresses in the deck is reached. An acceptable pour sequence for a multi-span bridge fitting the above criteria is shown in Figure 9.2.1.3. Since adjacent spans may not be poured within 72 hours of each other, the second pour is permitted to be the next most flexible section after the first pour. Note that the third and fourth pours require placement of both positive and negative moment sections. If the



**Figure 9.2.1.3**

contractor will be unable to complete the placement of the entire section in one pour, the positive moment area is to be placed first followed by the negative sections.

For superstructures which consist of a series of simply supported spans that require a deck placement sequence, locate transverse construction joints at the end of a span.

Where possible, consider orienting the direction of the concrete pours in the uphill direction, allowing gravity to reduce possible tension in the deck.

On bridges with strip seal expansion joints where a deck pour begins at the abutment, investigate the effects of beam end rotation. Too much rotation can negatively affect the joint size or cause deck and end block formwork to fail if not appropriately handled during concrete placement. If this is a problem, consider revising the pour sequence or placing the abutment end block after the deck is complete.

In all cases, a minimum of 72 hours is required between adjacent deck pours.

For unusual span length configurations, discuss the deck placement sequence with the Regional Bridge Construction Engineer.

**Design Assumptions for Deck Reinforcement Tables 9.2.1.1 and 9.2.1.2:**

- Transverse reinforcement is based on the traditional deck design method.
- Concrete strength,  $f'_c = 4$  ksi.
- Epoxy coated steel rebar strength,  $f_y = 60$  ksi.
- Dead load includes deck self-weight plus a future wearing course of 0.020 ksf. A load factor of 1.25 was applied to the future wearing course. Dead load bending moment is based on equation  $M_{DC} = w_{DC} \cdot L^2/10$ .
- Positive live load moments were taken from LRFD Table A4-1.
- Negative live load moments were interpolated from values in LRFD Table A4-1.
- Concrete clear cover for top transverse reinforcement is 3".
- Concrete clear cover for bottom transverse reinforcement is 1".
- For decks without a wearing course, 1/2" wear was assumed in determining the structural depth,  $d_s$ , for the bottom transverse reinforcement.
- For decks with a wearing course, the 2" wearing course was not used (sacrificial) in determining structural depth,  $d_s$ , for the bottom transverse reinforcement.
- Crack control used a Class 2 exposure condition ( $\gamma_e = 0.75$ ).
- For crack control check, although the actual concrete clear cover to top transverse bars exceeds 2 inches,  $d_c$  calculation used a maximum clear concrete cover equal to 2 inches.
- LRFD Art. 9.7.2.4 (under empirical design) requires that the ratio of the effective beam spacing to slab thickness be less than 18. The slab thicknesses given in the tables fit these requirements and are similar to what MnDOT has used successfully in the past.
- Bottom longitudinal reinforcement is distribution reinforcement per LRFD Article 9.7.3.2.

**EPOXY COATED STEEL REINFORCEMENT FOR DECKS SUPPORTED ON  
PRECAST PRETENSIONED CONCRETE BEAMS**

Negative moment design section is assumed at 10 inches from centerline for I-beams, based on 1/3 of the M-series beam top flange per LRFD Article 4.6.2.1.6 (conservative for MN-series and MW-series beams). Similarly, negative moment design section is assumed at 8.7 inches from centerline for rectangular beams, which is based on 1/3 of the rectangular beam width.

Maximum Beam Spacing <sup>①</sup>	Transverse Reinforcement Size and Spacing				Deck Thickness T <sup>②</sup>	Longitudinal Reinforcement Size and Spacing, Bottom Mat <sup>③</sup>	Longitudinal Reinforcement Size and Spacing, Top Mat <sup>③</sup>
	Bottom Mat		Top Mat				
	With Wearing Course	Without Wearing Course	Deck on I-Beam	Deck on Rectangular Beam			
5'-0"	4 @ 6.5"	4 @ 9"	4 @ 10"	4 @ 9.5"	9"	4 @ 9"	4 @ 1'-6"
5'-6"	4 @ 6.5"	4 @ 8.5"	4 @ 9"	4 @ 8.5"	9"	4 @ 9"	4 @ 1'-6"
6'-0"	4 @ 6.5"	4 @ 8"	4 @ 8.5"	4 @ 8"	9"	4 @ 9"	4 @ 1'-6"
6'-6"	4 @ 6"	4 @ 8"	4 @ 8"	4 @ 7.5"	9"	4 @ 9"	4 @ 1'-6"
7'-0"	4 @ 5.5"	4 @ 7.5"	4 @ 7.5"	4 @ 7"	9"	4 @ 8"	4 @ 1'-6"
7'-6"	4 @ 5.5"	4 @ 7"	4 @ 7"	4 @ 6.5"	9"	4 @ 8"	4 @ 1'-6"
8'-0"	4 @ 5"	4 @ 6.5"	4 @ 6.5"	4 @ 6.5"	9"	4 @ 7"	4 @ 1'-6"
8'-6"	5 @ 7.5"	4 @ 6"	4 @ 6.5"	4 @ 6"	9"	4 @ 7"	4 @ 1'-6"
9'-0"	5 @ 7"	4 @ 6"	4 @ 6"	4 @ 6"	9"	5 @ 10"	4 @ 1'-6"
9'-6"	5 @ 6.5"	4 @ 5.5"	4 @ 6"	4 @ 5.5"	9"	5 @ 9"	4 @ 1'-6"
10'-0"	5 @ 6"	4 @ 5.5"	4 @ 5.5"	4 @ 5"	9"	5 @ 8"	4 @ 1'-6"
10'-6"	5 @ 6"	4 @ 5"	4 @ 5"	5 @ 6.5"	9"	5 @ 8"	4 @ 1'-6"
11'-0"	5 @ 5.5"	5 @ 7.5"	5 @ 6"	5 @ 6"	9"	5 @ 8"	4 @ 1'-6"
11'-6"	5 @ 5.5"	5 @ 7"	5 @ 5.5"	5 @ 5.5"	9"	5 @ 8"	4 @ 1'-6"
12'-0"	5 @ 5"	5 @ 6.5"	5 @ 5.5"	5 @ 5.5"	9"	5 @ 7"	4 @ 1'-6"
12'-6"	6 @ 7"	5 @ 6.5"	5 @ 5"	5 @ 5"	9"	5 @ 7"	4 @ 1'-6"
13'-0"	6 @ 7"	5 @ 6.5"	5 @ 5"	5 @ 5"	9.5"	5 @ 7"	4 @ 1'-6"
13'-6"	6 @ 7.5"	5 @ 6.5"	5 @ 5"	5 @ 5"	9.75"	5 @ 8"	4 @ 1'-6"
14'-0"	6 @ 7"	5 @ 6.5"	5 @ 5"	6 @ 6"	10"	5 @ 8"	4 @ 1'-6"
14'-6"	6 @ 7.5"	5 @ 6.5"	5 @ 5"	6 @ 6"	10.25"	5 @ 8"	4 @ 1'-6"
15'-0"	6 @ 7.5"	5 @ 6.5"	5 @ 5"	6 @ 6"	10.5"	5 @ 8"	4 @ 1'-6"

① For skews ≤ 20°, beam spacing is measured along the skew.  
For skews > 20°, beam spacing is measured normal to roadway centerline.

② Deck thickness includes wearing course.

③ Reinforcement shown is for deck regions in non-pier areas only and is based on LRFD 5.10.6. Note that additional reinforcement is required for deck regions over/near piers. See Figure 9.2.1.8 for additional top longitudinal reinforcement required in deck regions over/near piers when only deck is continuous. For beams made continuous, design longitudinal reinforcement in deck regions over/near piers for factored negative moment.

**Table 9.2.1.1**

**EPOXY COATED STEEL REINFORCEMENT FOR DECKS SUPPORTED ON STEEL BEAMS**

Negative moment design section is assumed at 3 inches from centerline of beam, based on 1/4 of a 12 inch top flange per LRFD Article 4.6.2.1.6.

Maximum Beam Spacing <sup>①</sup>	Transverse Reinforcement Size and Spacing			Deck Thickness T <sup>②</sup>	Longitudinal Reinforcement Size and Spacing, Bottom Mat <sup>③</sup>	Longitudinal Reinforcement Size and Spacing, Top Mat <sup>③</sup>
	Bottom Mat		Top Mat			
	With Wearing Course	Without Wearing Course				
5'-0"	4 @ 6.5"	4 @ 9"	4 @ 8"	9"	4 @ 9"	4 @ 1'-6"
5'-6"	4 @ 6.5"	4 @ 8.5"	4 @ 7"	9"	4 @ 9"	4 @ 1'-6"
6'-0"	4 @ 6.5"	4 @ 8"	4 @ 6.5"	9"	4 @ 9"	4 @ 1'-6"
6'-6"	4 @ 6"	4 @ 8"	4 @ 6"	9"	4 @ 9"	4 @ 1'-6"
7'-0"	4 @ 5.5"	4 @ 7.5"	4 @ 5.5"	9"	4 @ 8"	4 @ 1'-6"
7'-6"	4 @ 5.5"	4 @ 7"	4 @ 5.5"	9"	4 @ 8"	4 @ 1'-6"
8'-0"	4 @ 5"	4 @ 6.5"	4 @ 5"	9"	4 @ 7"	4 @ 1'-6"
8'-6"	5 @ 7.5"	4 @ 6"	5 @ 6.5"	9"	4 @ 7"	4 @ 1'-6"
9'-0"	5 @ 7"	4 @ 6"	5 @ 6.5"	9"	4 @ 6"	4 @ 1'-6"
9'-6"	5 @ 6.5"	4 @ 5.5"	5 @ 6"	9"	4 @ 6"	4 @ 1'-6"
10'-0"	5 @ 6"	4 @ 5.5"	5 @ 6"	9"	4 @ 5"	4 @ 1'-6"
10'-6"	5 @ 6"	4 @ 5"	5 @ 5.5"	9"	4 @ 5"	4 @ 1'-6"
11'-0"	5 @ 6"	4 @ 5"	5 @ 5.5"	9.25"	4 @ 5"	4 @ 1'-6"
11'-6"	5 @ 6"	4 @ 5"	5 @ 5"	9.5"	4 @ 5"	4 @ 1'-6"
12'-0"	5 @ 6"	5 @ 7.5"	5 @ 5"	9.75"	4 @ 6"	4 @ 1'-6"
12'-6"	5 @ 6"	5 @ 7.5"	5 @ 5"	10"	4 @ 6"	4 @ 1'-6"
13'-0"	5 @ 6"	5 @ 7.5"	6 @ 6.5"	10.25"	4 @ 6"	4 @ 1'-6"
13'-6"	5 @ 6"	5 @ 7"	6 @ 6.5"	10.5"	4 @ 6"	4 @ 1'-6"
14'-0"	5 @ 6"	5 @ 7"	6 @ 6.5"	10.75"	4 @ 6"	4 @ 1'-6"
14'-6"	5 @ 5.5"	5 @ 7"	6 @ 6.5"	11"	4 @ 6"	4 @ 1'-6"
15'-0"	5 @ 5.5"	5 @ 7"	6 @ 6.5"	11.25"	4 @ 6"	4 @ 1'-6"

① For skews ≤ 20°, beam spacing is measured along the skew.

For skews > 20°, beam spacing is measured normal to roadway centerline.

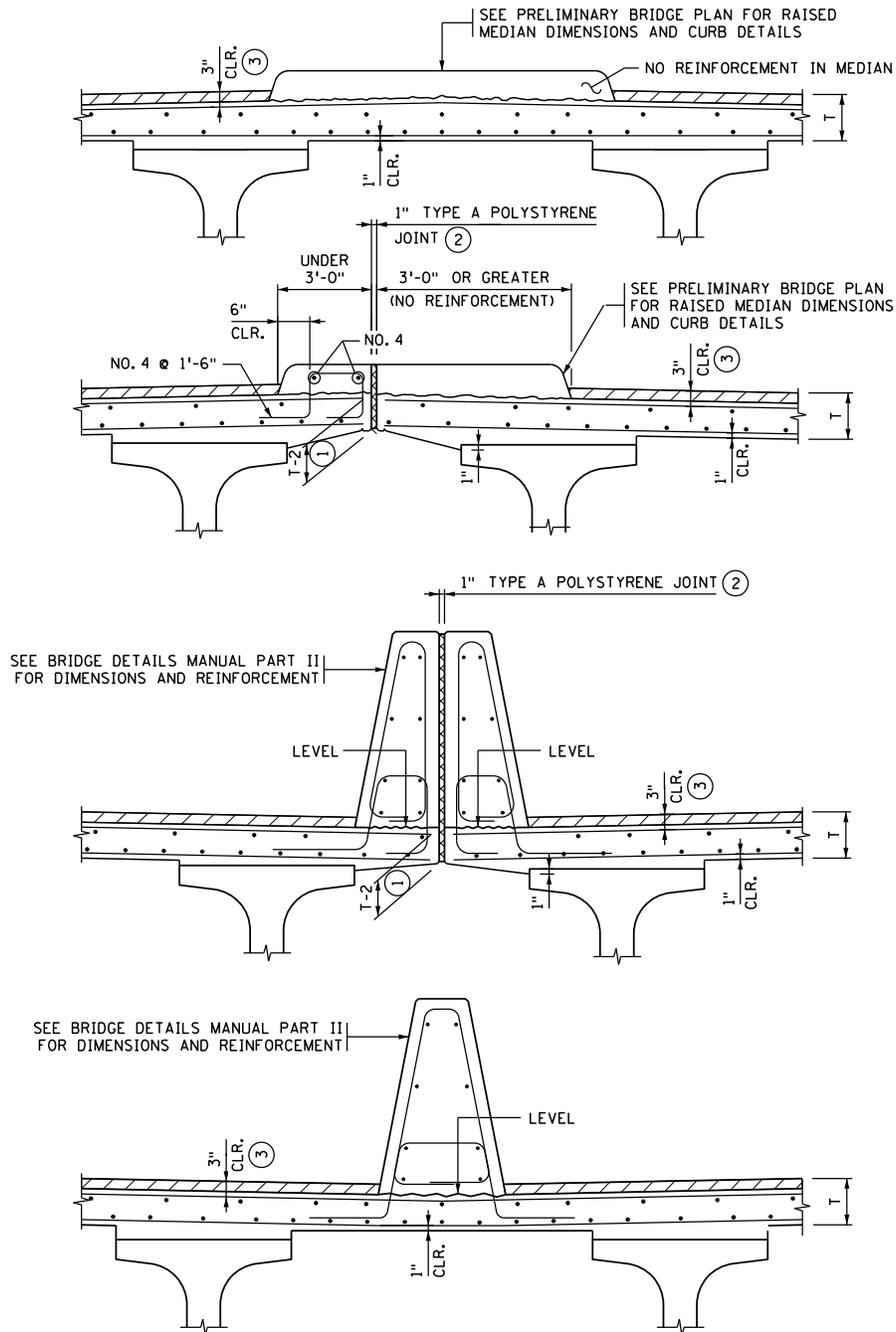
② Deck thickness includes wearing course.

③ Reinforcement shown is for positive moment region only and is based on LRFD 5.10.6. Where deck and steel beams are continuous, design longitudinal reinforcement in negative moment regions for the factored negative moment and meet requirements of LRFD Article 6.10.1.7. See Figure 9.2.1.9 for longitudinal reinforcing requirements in negative moment regions.

**Table 9.2.1.2**







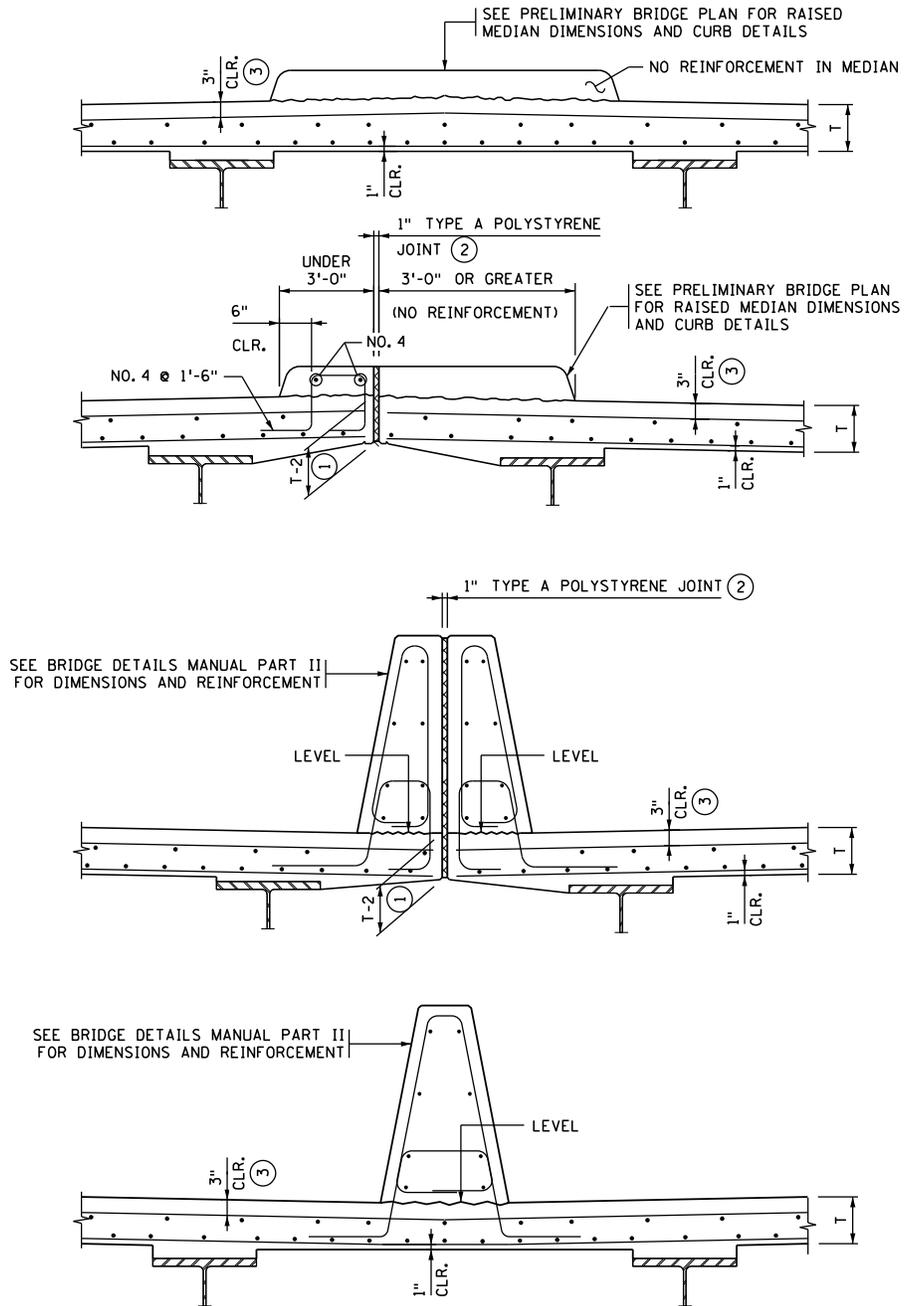
**NOTES:**

IF BOTH SHOULDER AREAS SLOPE INTO BARRIER BOTH ADJACENT BEAMS MAY HAVE TO DROP TO PREVENT NEGATIVE STOOLS.

- (1) T EQUALS DECK THICKNESS GIVEN IN TABLE 9.2.1.1. OR 9.2.1.2 OF THIS MANUAL.
- (2) PROVIDE SPLIT MEDIAN CAP WHEN REQUIRED BY REGIONAL BRIDGE CONSTRUCTION ENGINEER.
- (3) CONCRETE CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER FOR OTHER BARS MAY DIFFER.

**CONCRETE DECK REINFORCEMENT SECTIONS**  
(WITH CONCRETE WEARING COURSE)

**Figure 9.2.1.6**



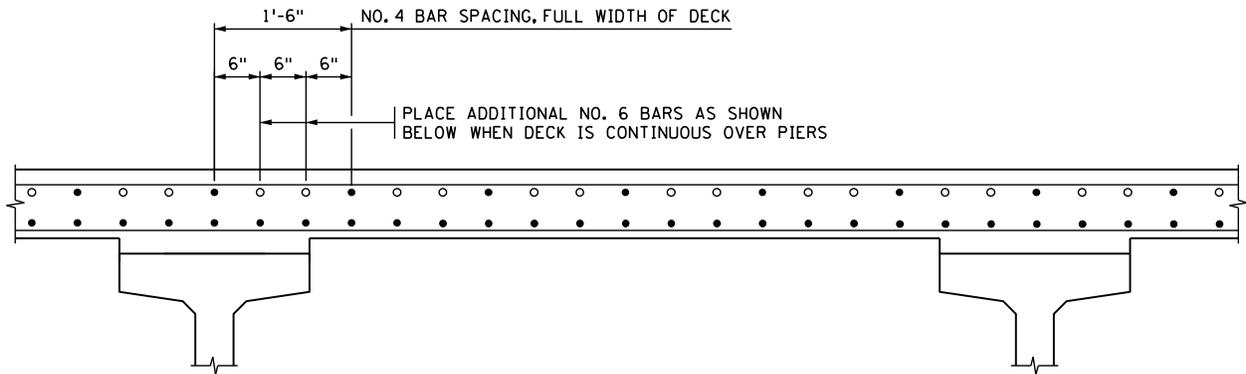
**NOTES:**

IF BOTH SHOULDER AREAS SLOPE INTO BARRIER BOTH ADJACENT BEAMS MAY HAVE TO DROP TO PREVENT NEGATIVE STOOLS.

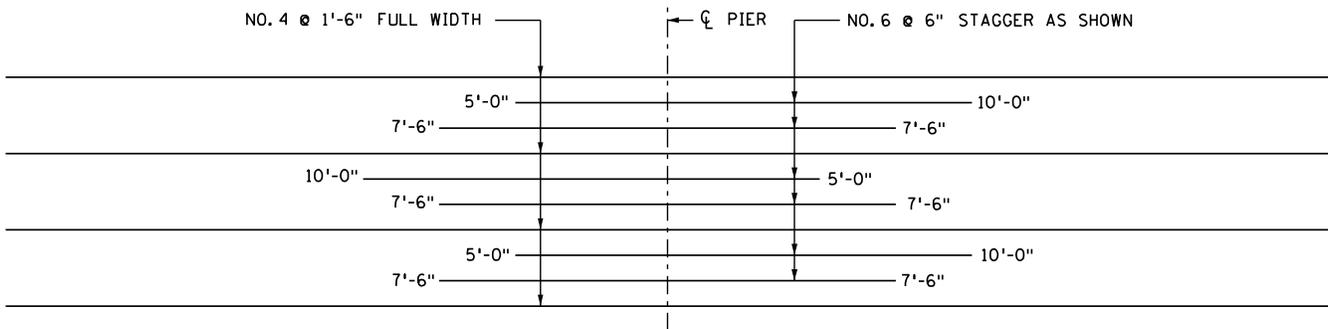
- ① T EQUALS DECK THICKNESS GIVEN IN TABLE 9.2.1.1. OR 9.2.1.2 OF THIS MANUAL.
- ② PROVIDE SPLIT MEDIAN CAP WHEN REQUIRED BY REGIONAL BRIDGE CONSTRUCTION ENGINEER.
- ③ CONCRETE CLEAR COVER SHOWN IS FOR EPOXY COATED STEEL BARS. COVER FOR OTHER BARS MAY DIFFER.

**CONCRETE DECK REINFORCEMENT SECTIONS**  
(WITH CONCRETE WEARING COURSE)

**Figure 9.2.1.7**



TRANSVERSE SECTION

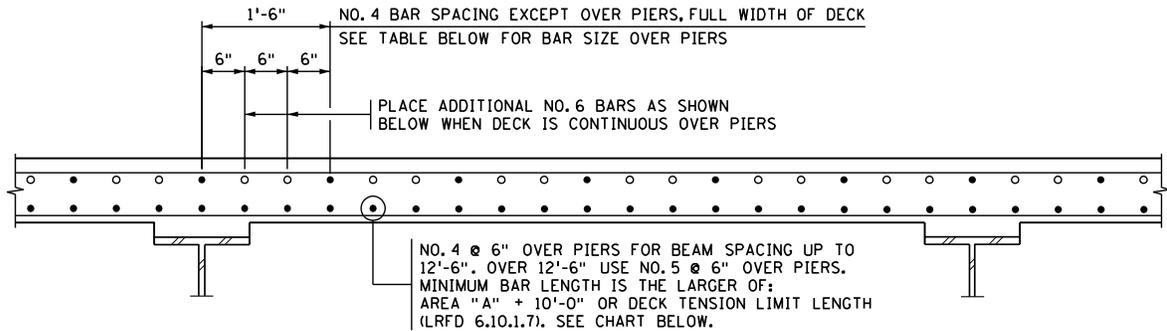


PLAN OF TOP LONGITUDINAL REINFORCEMENT AT PIER

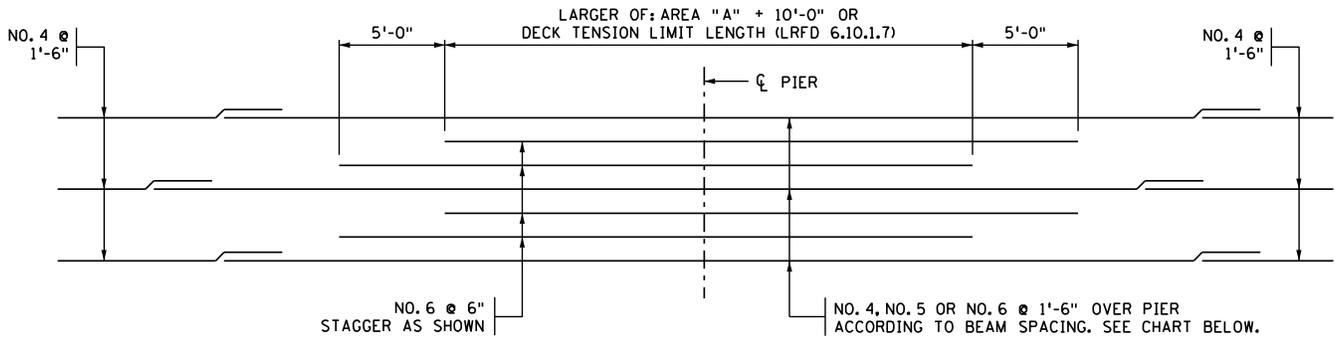
LONGITUDINAL REINFORCEMENT FOR CONCRETE DECK WITH MAIN REINFORCEMENT PERPENDICULAR TO TRAFFIC WITH ONLY DECK (NOT BEAMS) CONTINUOUS OVER PIER

CONCRETE DECK REINFORCEMENT DETAILS FOR PRESTRESSED CONCRETE BEAM SPANS

*Figure 9.2.1.8*



TRANSVERSE SECTION



PLAN OF TOP LONGITUDINAL REINFORCEMENT AT PIER

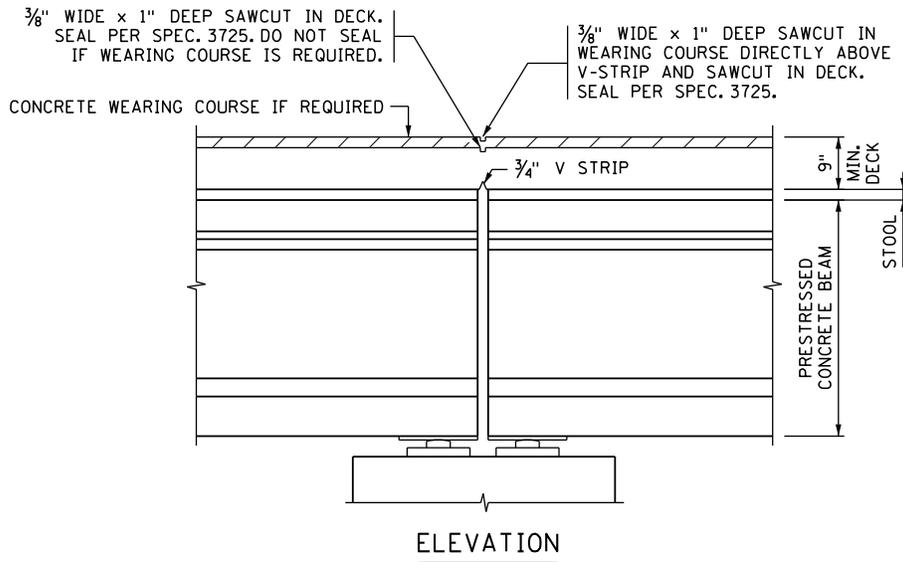
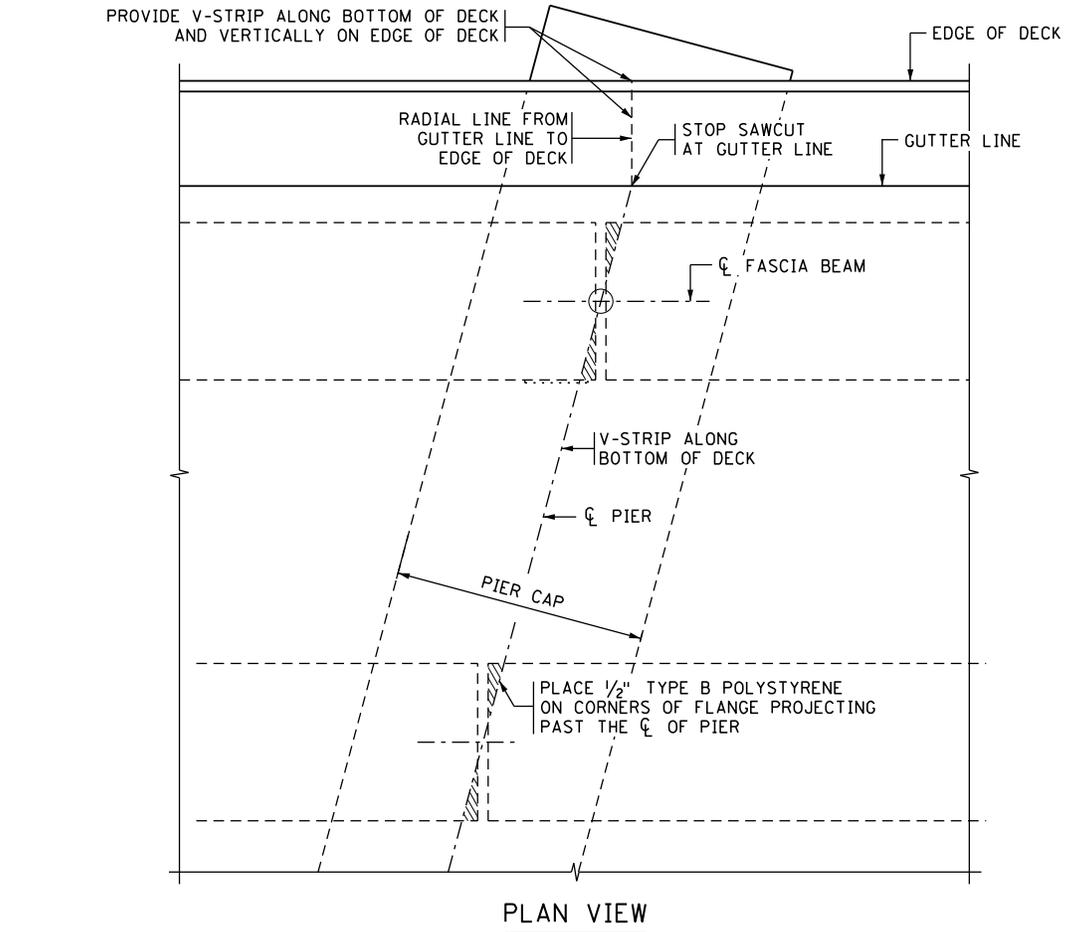
THROUGH REINFORCEMENT OVER PIERS		
BEAM SPACING	BOTTOM LONGITUDINAL	TOP LONGITUDINAL
UP TO 10'-6"	NO. 4 @ 6"	NO. 4 @ 1'-6"
OVER 10'-6" TO 12'-6"	NO. 4 @ 6"	NO. 5 @ 1'-6"
OVER 12'-6" TO 14'-6"	NO. 5 @ 6"	NO. 6 @ 1'-6"
OVER 14'-6"	SPECIAL DESIGN	

PERMISSIBLE SPLICES IN REINFORCEMENT BARS TO BE LOCATED AWAY FROM CL OF PIER AND ALTERNATED ON EACH SIDE OF PIER

LONGITUDINAL REINFORCEMENT FOR CONCRETE DECK WITH MAIN REINFORCEMENT PERPENDICULAR TO TRAFFIC AND CONTINUOUS OVER 3 OR MORE BEAMS

CONCRETE DECK REINFORCEMENT DETAILS FOR CONTINUOUS STEEL BEAM SPANS (AASHTO LRFD 6.10.1.7)

Figure 9.2.1.9



SAWCUT DETAIL AT PIERS  
(CONTINUOUS DECK OVER NON-CONTINUOUS PRESTRESSED BEAMS)

**Figure 9.2.1.10**

**9.3 Reinforced Concrete Deck Design Example**

This example demonstrates the design of a reinforced concrete deck supported on MN63 pretensioned concrete I-beams. The first part describes the design of the deck interior region (between the fascia beams) and the second part provides design procedures for the deck overhang region.

**[4.6.2.1]**

The deck is designed using the traditional approximate analysis method. The deck is assumed to carry traffic loads to the beam supports via one-way slab or beam action. The beams are parallel to the direction of traffic and the substructures are not skewed, so the primary reinforcement for the deck is placed perpendicular to the beams. Distribution steel is placed parallel to the beams.

**[9.7.3.2]**

The reinforced concrete deck section with wearing course is illustrated in Figure 9.3.1.

**A. Material and Design Parameters**

**[9.7.1.1]**

**[9.7.1.3]**

**Deck**

Unit weight of deck and wearing course (for loads),  $w_c = 0.150$  kcf

Unit weight of deck and wearing course (for  $E_c$ ),  $w_{cE} = 0.145$  kcf

Skew angle of bridge,  $\theta = 0$  degrees

Out-to-out bridge deck transverse width,  $b_{deck} = 52.00$  ft = 624 in

Weight of future wearing course,  $w_{fws} = 0.020$  kcf

Yield strength of reinforcing bars,  $f_y = 60$  ksi

Reinforcing bar modulus of elasticity,  $E_s = 29,000$  ksi

28 day concrete strength,  $f'_c = 4$  ksi

Center-to-center beam spacing,  $L_s = 9.00$  ft

Railing weight,  $w_{barrier} = 0.513$  klf (see Std. Figure 5-397.139(B))

Beam flange width,  $b_f = 34$  in (MN63 Prestressed I-Beam)

Deck overhang = 3.50 ft

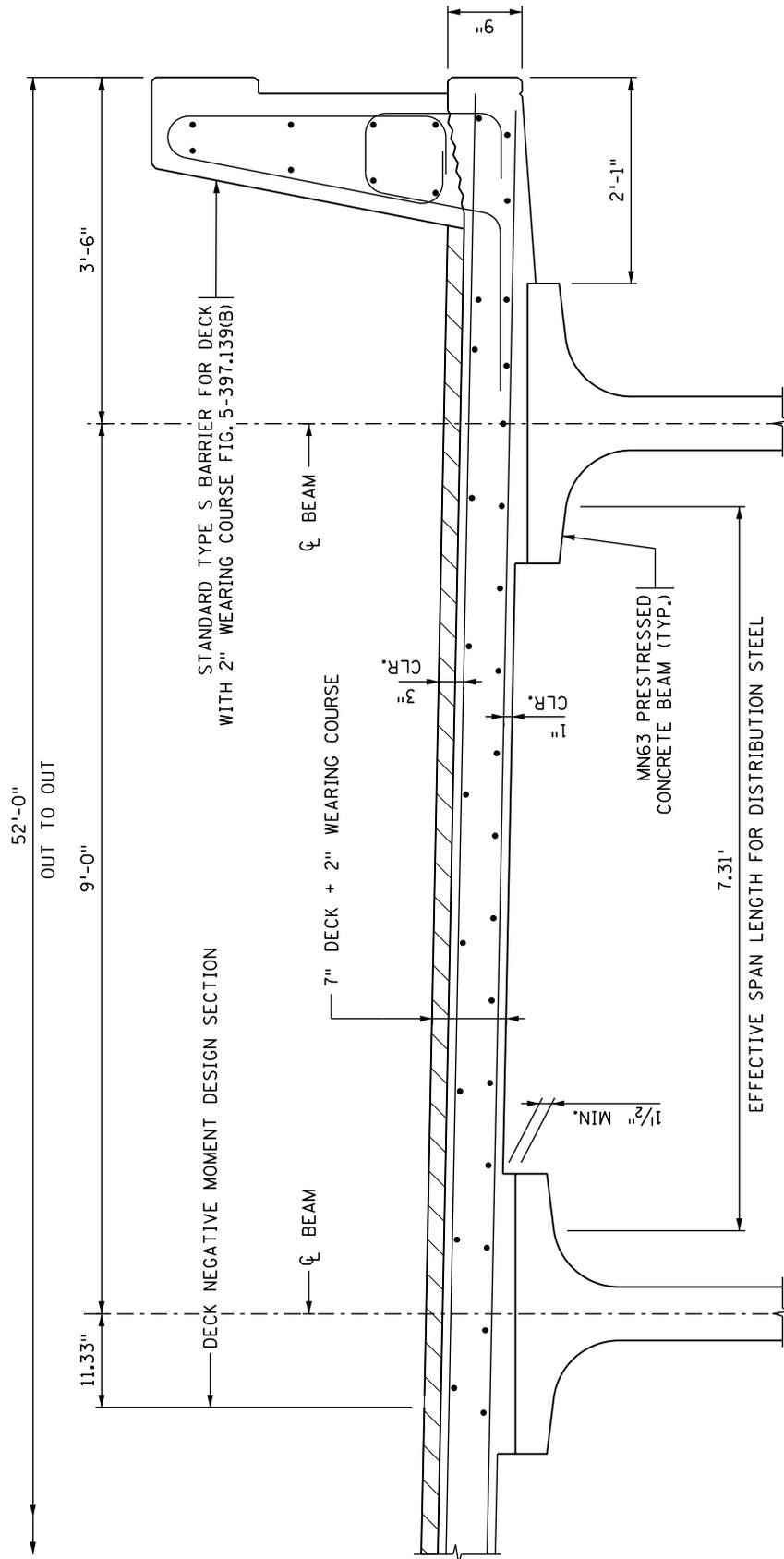
**B. Structural Analysis of Interior Region**

**[9.6.1]**

The deck is modeled as a continuous beam on pinned supports provided at the centerline of the supporting beams. The beams are assumed to be rigid, not permitting vertical movement. Recognizing that beams have top flanges that provide support for the deck over a finite dimension, the specifications permit designing negative moment reinforcement for locations that are offset from the centerline of the beam.

**[4.6.2.1.6]**

For prestressed beams, negative moments are checked at the design section located  $1/3$  of the flange width away from the beam centerline, with maximum offset of 15 inches. For the top flange width of 34 inches, check negative moments at a location 11.33 inches away from beam centerline. (Note that this differs from the design section chosen for the deck reinforcement tables, which are conservatively based on M-series beams with 30 inch flanges.) The design is based on a unit strip one foot wide.



PARTIAL DECK SECTION

Figure 9.3.1

**C. Live Loads**  
**[Appendix A4]**

The AASHTO LRFD Specifications contain tables listing the design live load moments (positive and negative) for decks supported on different beam spacings. The tabularized moments are for a one foot wide strip.

The limitations for use of the tables include a check on the overhang dimension. A minimum of 1.75 feet from the centerline of the fascia beam is permitted. The maximum overhang permitted,  $L_{ohmax}$ , is the lesser of:

$$L_{ohmax} = 6.00 \text{ ft}$$

or

$$L_{ohmax} = 0.625 \cdot L_s = 0.625 \cdot 9.00 = 5.63 \text{ ft} \quad \text{GOVERNS}$$

For this example the overhang check falls within the limits:

$$1.75 \text{ ft} < 3.50 \text{ ft} < 5.63 \text{ ft} \quad \text{OK}$$

The overhang dimension checks are satisfied, as are all other parameters specified for use of the design live load moment tables.

**Interpolate Design Live Load Moments**

LRFD Table A4-1 lists the following design live load moments for a beam spacing of 9.0 ft:

$$\text{Positive moment} = 6.29 \text{ kip-ft}$$

$$\text{Negative moment (9 in)} = 4.28 \text{ kip-ft}$$

$$\text{Negative moment (12 in)} = 3.71 \text{ kip-ft}$$

Interpolate to obtain the negative moment at the design section (11.33 inches away from the center of the supporting beam):

$$M_{LL(neg)} = 4.28 - \left( \frac{11.33 - 9}{12 - 9} \right) \cdot (4.28 - 3.71) = 3.84 \text{ kip-ft}$$

The values in LRFD Table A4 -1 include the multiple presence and dynamic load allowance factors.

**D. Dead Loads**

The dead load moments are based on the self-weight of the 7 inch partial depth deck, the 2 inch wearing course, and a 0.020 ksf future wearing surface.

$$\text{Depth of concrete deck, } d_{deck} = 7 + 2 = 9 \text{ in}$$

Dead loads will be computed for a strip of deck 1 foot wide. MnDOT practice is to simplify the dead load bending moment calculations by computing both the positive and negative dead load bending moments using:

$$M_{DC} = \frac{W_{DC} \cdot L_s^2}{10}$$

Deck and Wearing Course Load:

$$W_{deck} = w_c \cdot d_{deck} = (0.150) \cdot 9 \cdot \frac{1}{12} = 0.11 \text{ klf}$$

Future Wearing Surface Load:

$$W_{fws} = 0.02 \text{ klf}$$

Combined Dead Load:

$$W_{DC} = W_{deck} + W_{fws} = 0.11 + 0.02 = 0.13 \text{ klf}$$

Dead Load Bending Moment:

$$M_{DC} = \frac{0.13 \cdot 9^2}{10} = 1.05 \text{ kip-ft}$$

**E. Flexural Design Moments**  
**[1.3.3 – 1.3.5]**

The load modifiers for the deck design are:

$$\begin{aligned} \eta_D &= 1.00 \\ \eta_R &= 1.00 \\ \eta_I &= 1.00 \end{aligned}$$

$$\text{Then } \eta_{cum} = \eta_D \cdot \eta_R \cdot \eta_I = 1.00$$

**[Table 3.4.1-1]**

Use the load factors provided in LRFD Article 3.4.1 to generate the Strength I and Service I design moments.

**Strength I Limit State Loads**

$$U_1 = \eta_{cum} \cdot (1.25 \cdot DC + 1.75 \cdot LL)$$

Negative Design Moment:

$$M_{u(neg)} = 1.00 \cdot [1.25 \cdot (1.05) + 1.75 \cdot (3.84)] = 8.03 \text{ kip-ft}$$

Positive Design Moment:

$$M_{u(pos)} = 1.00 \cdot [1.25 \cdot (1.05) + 1.75 \cdot (6.29)] = 12.32 \text{ kip-ft}$$

**Service I Limit State Loads**

$$S_1 = \eta_{cum} \cdot (1.0 \cdot DC + 1.0 \cdot LL)$$

Negative Design Moment:

$$M_{s(neg)} = 1.00 \cdot [1.0 \cdot (1.05) + 1.0 \cdot (3.84)] = 4.89 \text{ kip-ft}$$

Positive Design Moment:

$$M_{s(\text{pos})} = 1.00 \cdot [1.0 \cdot (1.05) + 1.0 \cdot (6.29)] = 7.34 \text{ kip-ft}$$

**F. Top Steel  
(Negative  
Moment)**

[5.6.3]

**Flexure Strength Check**

The top reinforcement has a clear cover of 3 inches (which includes the 2 inch wearing course). Design the negative moment reinforcement assuming a singly reinforced cross section.

[5.5.4.2]

Assume the section is tension-controlled and the flexural resistance factor,  $\phi = 0.90$ .

Based on BDM Table 9.2.1.1, try #4 bars with a 6 inch center-to-center spacing.

Determine depth,  $d_s$ , from extreme compression fiber to tension reinforcement.

$$d_s = d_{\text{deck}} - \text{cover} - \frac{1}{2} \cdot d_b = 9 - 3 - \frac{1}{2} \cdot 0.5 = 5.75 \text{ in}$$

Width of compression face of member,  $b = 12 \text{ in}$

Area of top steel provided is:

$$A_{s(\text{top})} = A_b \cdot \left( \frac{12}{\text{bar spacing}} \right) = 0.20 \cdot \left( \frac{12}{6} \right) = 0.40 \frac{\text{in}^2}{\text{ft}}$$

Then:

$$a = c \cdot \beta_1 = \frac{A_{s(\text{top})} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.40 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.59 \text{ in}$$

$$\begin{aligned} \phi \cdot M_n &= \phi \cdot A_{s(\text{top})} \cdot f_y \cdot \left( d_s - \frac{a}{2} \right) = 0.9 \cdot 0.40 \cdot 60 \cdot \left( 5.75 - \frac{0.59}{2} \right) \cdot \frac{1}{12} \\ &= 9.82 \text{ kip-ft} > 8.03 \text{ kip-ft} \quad \text{OK} \end{aligned}$$

[5.5.4.2]

Validate the assumption of 0.9 for resistance factor:

Calculate the depth of the section in compression:

$$c = \frac{a}{\beta_1} = \frac{0.59}{0.85} = 0.69 \text{ in}$$

[5.6.2.1]

Concrete compression strain limit  $\epsilon_c = 0.003$

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d_s - c) \cdot \left(\frac{\epsilon_c}{c}\right) = (5.75 - 0.69) \cdot \left(\frac{0.003}{0.69}\right) = 0.0220 > \epsilon_{tl} = 0.005$$

Therefore,  $\phi = 0.9$

[5.6.7]

**Crack Control Check**

The LRFD crack control check places a limit on the spacing of reinforcement to prevent severe and excessive flexural cracking. This is accomplished by limiting the spacing of reinforcing bars as follows:

$$s \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Also, the stress in the reinforcement,  $f_{ss}$ , is limited to:

$$f_{ss} \leq 0.6 \cdot f_y = 0.6 \cdot 60 = 36.0 \text{ ksi}$$

Per Article 5.3.2 of this manual, use a maximum clear cover of 2.0 inches to compute  $d_c$ . Assuming #4 bars are used:

$$d_c = 2.0 + 0.5 \cdot d_b = 2.0 + 0.5 \cdot 0.50 = 2.25 \text{ in}$$

The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. To simplify the calculation, the section is assumed to be singly reinforced.

[5.4.2.4 & 5.6.1]

Referring to Figure 9.3.2, determine the distance,  $x$ , from the bottom of the deck to the neutral axis:

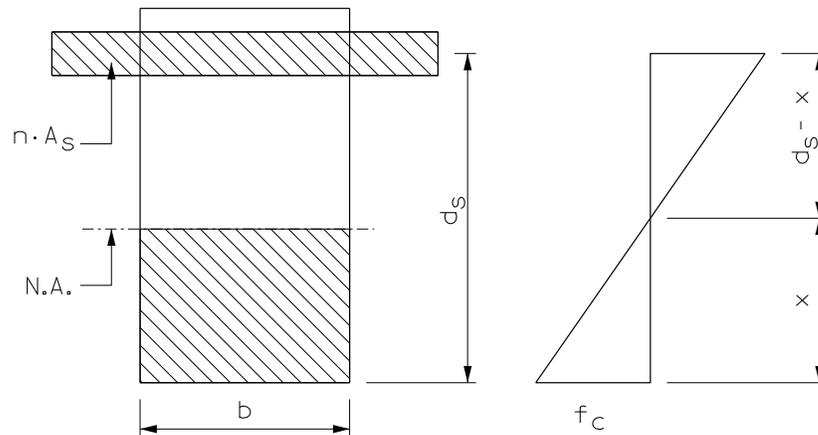


Figure 9.3.2

$$n = \frac{E_s}{E_c} = \frac{29,000}{120,000 \cdot K_1 \cdot w_{CE}^2 \cdot f_c^{0.33}} = \frac{29,000}{120,000 \cdot 1.0 \cdot 0.145^2 \cdot 4^{0.33}} = 7.27$$

$$n \cdot A_s = 7.27 \cdot 0.40 = 2.908$$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d_s - x)$$

$$\frac{12 \cdot x^2}{2} = 2.908 \cdot (5.75 - x)$$

solving,  $x = 1.44$  in

Determine the lever arm between service load flexural force components:

$$j \cdot d_s = d_s - \frac{x}{3} = 5.75 - \frac{1.44}{3} = 5.27 \text{ in}$$

The stress in the reinforcement when subjected to the Service I moment is:

$$f_{ss} = \frac{M_{s(neg)}}{A_s \cdot j \cdot d_s} = \frac{4.89 \cdot 12}{0.40 \cdot 5.27} = 27.8 \text{ ksi} < 36.0 \text{ ksi} \quad \text{OK}$$

Find  $\beta_s$ . For determination of deck depth,  $h$ , conservatively assume 0.5 inches of wear:

$$h = d_{deck} - 0.5 = 9 - 0.5 = 8.5 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.25}{0.7 \cdot (8.5 - 2.25)} = 1.51$$

For severe exposure, use  $\gamma_e=0.75$ . Then, maximum bar spacing,  $s_{max}$ , is:

$$s_{max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.51 \cdot 27.8} - 2 \cdot 2.25 = 8.01 \text{ in} > 6 \text{ in} \quad \text{OK}$$

**[5.6.3.3]**

**Minimum Reinforcement**

Check that reinforcement can carry the smaller of:

- Cracking moment,  $M_{cr}$
- $1.33 \cdot M_{u(neg)}$

Conservatively assume the full 9 inch deep section for the minimum reinforcement check:

$$S_{deck} = \frac{b \cdot (d_{deck})^2}{6} = \frac{12 \cdot 9^2}{6} = 162 \text{ in}^3$$

**[5.4.2.6]**

Determine the modulus of rupture,  $f_r$ :

$$\lambda = 1.0 \text{ for normal weight concrete}$$

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{4} = 0.48 \text{ ksi}$$

Take  $\gamma_1 = 1.60$  and  $\gamma_3 = 0.67$  for ASTM Grade 60 reinforcement.

Combining these parameters results in a cracking moment,  $M_{cr}$ , of:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_{deck} = 0.67 \cdot 1.60 \cdot 0.48 \cdot 162 \cdot \frac{1}{12} = 6.95 \text{ kip-ft}$$

Compare this to:

$$1.33 \cdot M_{u(neg)} = 1.33 \cdot 8.03 = 10.68 \text{ kip-ft} > 6.95 \text{ kip-ft}$$

Use the  $M_{cr}$  value to check minimum reinforcement.

$$\phi \cdot M_n = 9.82 \text{ kip-ft} > 6.95 \text{ kip-ft} \quad \text{OK}$$

**G. Bottom Steel  
(Positive Moment)  
[5.6.3]**

**Flexure Strength Check**

The bottom reinforcement has a clear cover of one inch. Because the wearing course may be removed in future milling operations, do not include it in structural capacity computations. Design the positive moment reinforcement assuming a singly reinforced cross section.

**[5.5.4.2]**

Assume the section is controlled in tension and the flexural resistance factor,  $\phi = 0.90$ .

Based on BDM Table 9.2.1.1, try #5 bars with a 7 inch center-to-center spacing.

Determine the depth,  $d_s$ , from extreme compression fiber to tension reinforcement.

$$d_s = d_{deck} - \text{cover} - \text{wear course} - \frac{1}{2} \cdot d_b = 9 - 1 - 2 - \frac{1}{2} \cdot 0.63 = 5.69 \text{ in}$$

Width of compression face of member,  $b = 12 \text{ in}$

Area of top steel provided is:

$$A_{s(\text{bot})} = A_b \cdot \left( \frac{12}{\text{bar spacing}} \right) = 0.31 \cdot \left( \frac{12}{7} \right) = 0.53 \frac{\text{in}^2}{\text{ft}}$$

Then:

$$a = c \cdot \beta_1 = \frac{A_{s(\text{bot})} \cdot f_y}{0.85 \cdot f_c \cdot b} = \frac{0.53 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.78 \text{ in}$$

$$\begin{aligned} \phi \cdot M_n &= \phi \cdot A_{s(\text{bot})} \cdot f_y \cdot \left( d_s - \frac{a}{2} \right) = 0.9 \cdot 0.53 \cdot 60 \cdot \left( 5.69 - \frac{0.78}{2} \right) \cdot \frac{1}{12} \\ &= 12.64 \text{ kip-ft} > 12.32 \text{ kip-ft} \quad \text{OK} \end{aligned}$$

#### [5.5.4.2]

Validate the assumption of 0.9 for resistance factor:

Calculate the depth of the section in compression:

$$c = \frac{a}{\beta_1} = \frac{0.78}{0.85} = 0.92 \text{ in}$$

Concrete compression strain limit  $\epsilon_c = 0.003$

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d_s - c) \cdot \left( \frac{\epsilon_c}{c} \right) = (5.69 - 0.92) \cdot \left( \frac{0.003}{0.92} \right) = 0.0156 > \epsilon_{tl} = 0.005$$

Therefore,  $\phi = 0.9$

#### [5.6.7]

##### Crack Control Check

As noted previously, the limit on spacing of reinforcement for crack control is:

$$s \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Also, the stress in the reinforcement,  $f_{ss}$ , is limited to:

$$f_{ss} \leq 0.6 \cdot f_y = 0.6 \cdot 60 = 36.0 \text{ ksi}$$

For #5 bars with 1 inch of cover,  $d_c$  is:

$$d_c = \text{cover} + 0.5 \cdot d_b = 1.00 + 0.5 \cdot 0.625 = 1.31 \text{ in}$$

Compute the stress in the reinforcement using a cracked section analysis of a singly reinforced section. Begin by locating the neutral axis.

As calculated previously,  $n = 7.27$

$$n \cdot A_s = 7.27 \cdot 0.53 = 3.853$$

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d_s - x)$$

$$\frac{12 \cdot x^2}{2} = 3.853 \cdot (5.69 - x)$$

solving,  $x = 1.62$  in

Determine the lever arm between service load flexural force components.

$$j \cdot d_s = d_s - \frac{x}{3} = 5.69 - \frac{1.62}{3} = 5.15 \text{ in}$$

The stress in the reinforcement when subjected to the Service I design moment is:

$$f_{ss} = \frac{M_{s(\text{pos})}}{A_s \cdot j \cdot d_s} = \frac{7.34 \cdot 12}{0.53 \cdot 5.15} = 32.3 \text{ ksi} < 36.0 \text{ ksi} \quad \text{OK}$$

Find  $\beta_s$ . For determination of deck depth,  $h$ , conservatively ignore the 2 inch wearing course:

$$h = d_{\text{deck}} - 2.0 = 9 - 2 = 7.0 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{1.31}{0.7 \cdot (7.0 - 1.31)} = 1.33$$

For severe exposure, use  $\gamma_e = 0.75$ . Then, maximum bar spacing,  $s_{\text{max}}$ , is:

$$s_{\text{max}} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.33 \cdot 32.3} - 2 \cdot 1.31 = 9.60 \text{ in} > 7 \text{ in} \quad \text{OK}$$

**[5.6.3.3]**

**Minimum Reinforcement Check**

Check that reinforcement can carry the smaller of:

- Cracking moment,  $M_{cr}$
- $1.33 \cdot M_{u(\text{neg})}$

Conservatively assuming the full 9 inch deep section for the minimum reinforcement check,  $S = 162 \text{ in}^3$  (previously calculated).

**[5.4.2.6]**

Also, the modulus of rupture,  $f_r = 0.48 \text{ ksi}$  (previously calculated)

Taking  $\gamma_1 = 1.60$  and  $\gamma_3 = 0.67$  for ASTM Grade 60 reinforcement, the cracking moment,  $M_{cr}$ , is:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_{deck} = 0.67 \cdot 1.60 \cdot 0.48 \cdot 162 \cdot \frac{1}{12} = 6.95 \text{ kip-ft}$$

Compare this to:

$$1.33 \cdot M_{u(pos)} = 1.33 \cdot 12.32 = 16.39 \text{ kip-ft} > 6.95 \text{ kip-ft}$$

Use the  $M_{cr}$  value to check minimum reinforcement.

$$\phi \cdot M_n = 12.65 \text{ kip-ft} > 6.95 \text{ kip-ft} \quad \text{OK}$$

**H. Bottom  
Longitudinal  
Reinforcement**

**[9.7.3.2]**

As part of the Traditional Design Method an “equivalent width method” for reinforced bridge deck designs is utilized. To ensure proper load distribution, reinforcement placed perpendicular to the primary reinforcement must be provided in the bottom mat. This reinforcement is a fraction of the primary steel required for positive moment. For decks where the primary reinforcement is placed perpendicular to traffic, the longitudinal reinforcement requirement in the bottom mat is:

$$PCT = \left( \frac{220}{\sqrt{S_e}} \right) \leq 67\%$$

where  $S_e$  is the effective span length in feet

**[9.7.2.3]**

The effective span length is a function of the beam spacing and type of beam. For prestressed concrete I-beam sections, the effective span length,  $S_e$ , is:

$$\begin{aligned} S_e &= \text{beam spacing} - \text{top flange width} + \text{one flange overhang} \\ &= 9.00 - \frac{34}{12} + \frac{13.75}{12} = 7.31 \text{ ft} \end{aligned}$$

$$PCT = \left( \frac{220}{\sqrt{S_e}} \right) = \left( \frac{220}{\sqrt{7.31}} \right) = 82.6\% \geq 67\%$$

Use 67% of the primary steel in the bottom mat.

The required area of steel is:

$$A_{s(req)} = 0.67 \cdot A_{s(bot)} = 0.67 \cdot 0.53 = 0.36 \text{ in}^2/\text{ft}$$

Try #5 bars on 10 inch centers. Area of steel provided equals:

$$A_{s(prov)} = A_b \cdot \left( \frac{12}{\text{spacing}} \right) = 0.31 \cdot \left( \frac{12}{10} \right) = 0.37 \frac{\text{in}^2}{\text{ft}} > 0.36 \frac{\text{in}^2}{\text{ft}} \quad \text{OK}$$

**I. Top Longitudinal Reinforcement**  
**[5.10.6]**

The top longitudinal bars must meet the shrinkage and temperature reinforcement requirements.

The least width  $b = b_{deck} = 624$  in

Take the least depth,  $h$ , as equal to the full deck thickness (conservative), which is 9 inches.

Then:

$$A_{stemp} \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.30 \cdot 624 \cdot 9}{2 \cdot (624 + 9) \cdot 60} = 0.096 \frac{\text{in}^2}{\text{ft}}$$

In addition:

$$0.11 \text{ in}^2/\text{ft} \leq A_{stemp} \leq 0.60 \text{ in}^2/\text{ft}$$

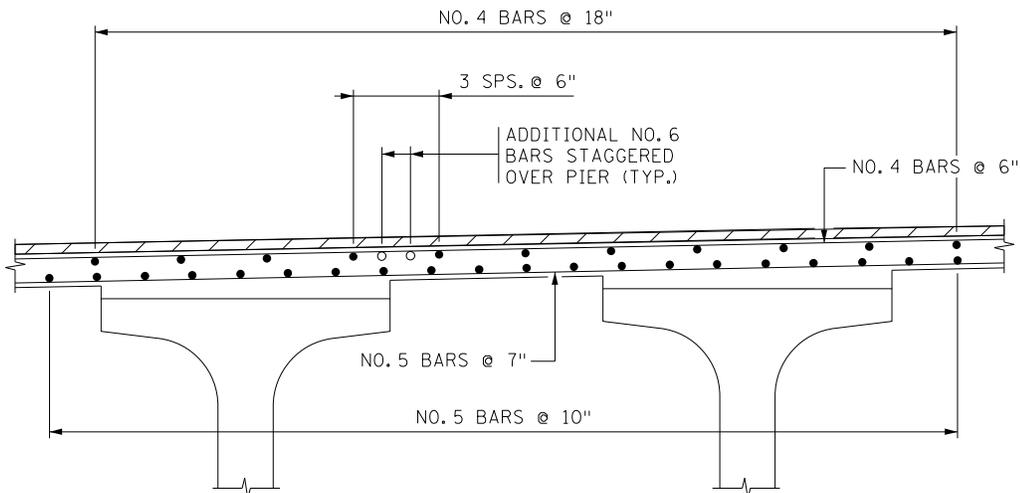
and

maximum bar spacing is 18 inches

Therefore, use #4 bars spaced at 18 inches ( $A_s = 0.13 \text{ in}^2/\text{ft}$ ) for the top longitudinal reinforcement.

MnDOT includes additional reinforcement over the piers when the deck is continuous, but the beams are not continuous. The additional reinforcing consists of two #6 bars placed on 6 inch centers between the top mat #4 bars. Refer to Figure 9.2.1.8 for typical reinforcement detailing.

Figure 9.3.3 illustrates the final reinforcement layout for the interior region of the deck.



**Figure 9.3.3**

**J. Structural Analysis of Deck Overhang Region [A13.4.1]**

Figure 9.3.4 illustrates the deck overhang region. Four cases must be considered for the deck overhang design:

- Case 1: Extreme Event II evaluated at the toe of the barrier for the dead load plus horizontal collision force.
- Case 2: Extreme Event II evaluated at the edge of the beam flange for the dead load plus horizontal collision force plus live load.
- Case 3: Strength I evaluated at the edge of the beam flange for the dead load plus live load.
- Case 4: Extreme Event II evaluated at the edge of the beam flange for the dead load plus vertical collision force plus live load.

For this example, the distance from the edge of flange to the gutter line is less than 1 foot, so a live load wheel load is not considered. Also, the dead load moment is a small fraction of the moment due to the collision load, so the higher Strength I load factor for dead load does not have an appreciable effect on the Strength I load combination when comparing it to Extreme Event II. Therefore, by inspection, Case 3 will not govern over Cases 1 and 2, so Case 3 calculations are not included in this example.

**K. Overhang Region Analysis, Case 1**

**Geometry and Loads**

Case 1 is Extreme Event II checked at the toe of the barrier for dead load and the horizontal collision force. Referring to Figure 9.3.4, determine the center of gravity location for the barrier by considering the area of a rectangular block that encompasses the entire barrier cross-section and subtracting components ①, ②, and ③. Results are shown in Table 9.3.1:

**Table 9.3.1 Determination of Barrier Center of Gravity Location**

Component Description	Width (in)	Height (in)	Area (in <sup>2</sup> )	Moment Arm From Barrier Toe (in)	Area · Moment Arm (in <sup>3</sup> )
Block encompassing barrier	18.38	38.00	698.44	9.19	6418.66
① (triangle)	7.38	38.00	-140.22	2.46	-344.94
② (rectangle)	2.00	25.00	-50.00	17.38	-869.00
③ (triangle)	16.38	2.00	-16.38	10.92	-178.87

Total = 491.84 in<sup>2</sup>

Total = 5025.85 in<sup>3</sup>

Then C.G. location,  $x_{cg}$ , from barrier toe is:  $x_{cg} = \frac{5025.85}{491.84} = 10.22$  in

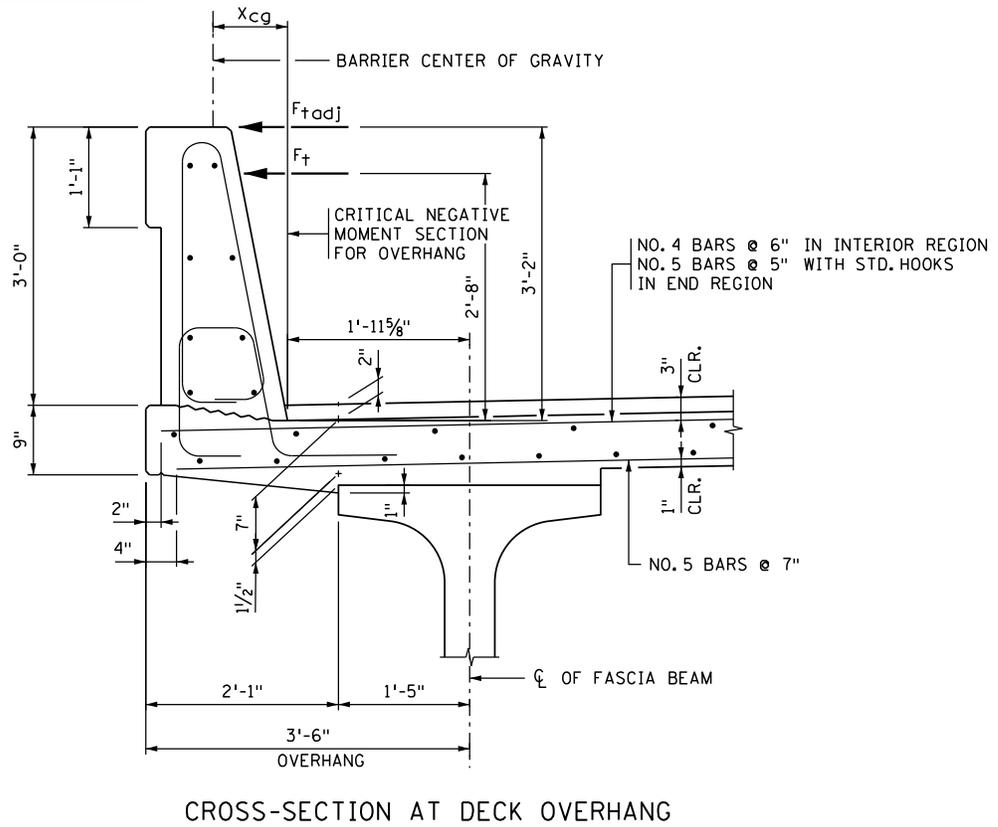
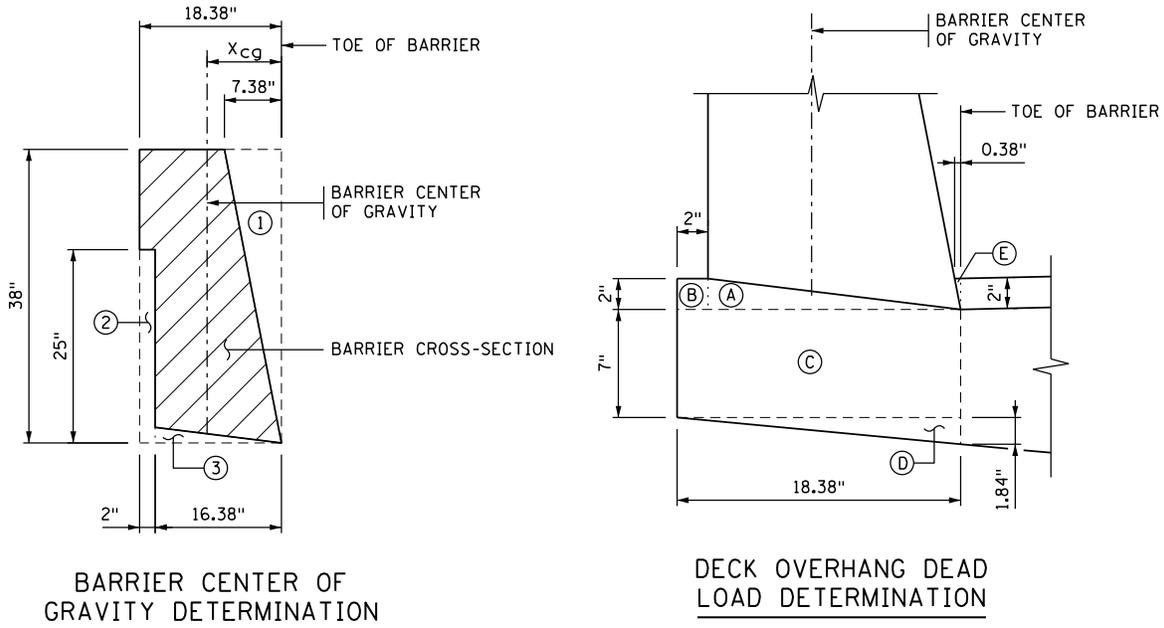


Figure 9.3.4

Overhang = 3.50 ft = 42 in

Distance from centerline of beam to edge of top flange = 17 in

Edge of deck to critical negative moment section at barrier toe,

$$L_{crit} = 18.38 \text{ in}$$

Deck thickness,  $h_{deck\_toe}$ , at barrier toe (ignoring wearing course):

$$h_{deck\_toe} = (9 - 2) + \left( \frac{18.38}{42 - 17} \right) \cdot [(2 + 7 + 1.5 + 1) - 9] = 8.84 \text{ in}$$

Referring again to Figure 9.3.4, determine dead load moments acting on the deck at the toe of barrier. Results are shown in Table 9.3.2:

**Table 9.3.2 Determination of Dead Load Moments at Barrier Toe for a 1 ft. Deck Strip Width**

Component Description	Width (in)	Height (in)	$w_{DC}$ (kips)	Moment Arm From Barrier Toe (in)	Unfactored Moment $M_{DC}$ (kip-ft)
Barrier			0.513	10.22	0.437
(A) (triangle)	16.38	2.00	0.017	10.92	0.015
(B) (rectangle)	2.00	2.00	0.004	17.38	0.006
(C) (rectangle)	18.38	7.00	0.134	9.19	0.103
(D) (triangle)	18.38	1.84	0.018	6.13	0.009
(E) (triangle)	0.38	2.00	0.000	0.13	0.000

Total  $w_{DC}$  = 0.686 kips for 1 ft strip width

Total  $M_{DC}$  = 0.570 kip-ft for a 1 ft strip width

**[A13.2]**

**Collision Force Tension and Bending Moment**

Using the yield line analysis method of LRFD Appendix A13, values for the nominal resistance,  $R_w$ , flexural resistance about the horizontal axis,  $M_c$ , and critical wall length,  $L_c$ , were calculated (not included here) for a 36 inch Type S barrier mounted on a deck that includes a wearing course (Std. Figure 5-397.139(B)):

Barrier int.:  $R_{w\_int} = 117.4 \text{ kips}$     $M_{c\_int} = 17.1 \text{ kip-ft/ft}$     $L_{c\_int} = 10.9 \text{ ft}$

Barrier end:  $R_{w\_end} = 71.8 \text{ kips}$     $M_{c\_end} = 22.8 \text{ kip-ft/ft}$     $L_{c\_end} = 5.0 \text{ ft}$

For a barrier meeting NCHRP Report 350 Test Level 4:

Transverse collision load  $F_t = 54 \text{ kips}$

Height of load application  $H_e = 32 \text{ in}$  (distance above top of wearing course)

Because the yield line equations in LRFD assume the collision load is applied at the top of the barrier, adjust  $F_t$  for the difference between the

barrier height and height of application. Refer to Figure 9.3.4. Note that the barrier sits on the partial depth deck with its toe 2 inches below the top of wearing course:

$$H_{\text{barrier}} = 38 \text{ in}$$

$$F_{\text{tadj}} = F_t \cdot \left( \frac{H_e + 2}{H_{\text{barrier}}} \right) = 54 \cdot \left( \frac{32 + 2}{38} \right) = 48.3 \text{ kips}$$

Because the barrier capacity can be excessively large compared to the collision load, MnDOT requires that the deck overhang be designed to resist a transverse collision force equal to the lesser of the barrier capacity  $R_w$  or  $\frac{4}{3} \cdot F_{\text{tadj}}$ :

$$F_{\text{coll\_int}} = R_{w\_int} = 117.4 \text{ kips}$$

$$F_{\text{coll\_end}} = R_{w\_end} = 71.8 \text{ kips}$$

or

$$= \frac{4}{3} \cdot F_{\text{tadj}} = \frac{4}{3} \cdot 48.3 = 64.4 \text{ kips} \quad \text{GOVERNS IN BOTH REGIONS}$$

Since  $R_{w\_int}$  and  $R_{w\_end}$  do not govern, the  $M_c$  values must also be adjusted to correspond with the collision load:

$$M_{\text{cadj\_int}} = \frac{F_{\text{coll\_int}}}{R_{w\_int}} \cdot M_{c\_int} = \frac{64.4}{117.4} \cdot 17.1 = 9.4 \frac{\text{kip-ft}}{\text{ft}}$$

$$M_{\text{cadj\_end}} = \frac{F_{\text{coll\_end}}}{R_{w\_end}} \cdot M_{c\_end} = \frac{64.4}{71.8} \cdot 22.8 = 20.5 \frac{\text{kip-ft}}{\text{ft}}$$

For deck overhang design, assume that the collision load is distributed over a length of  $L_{c\_int} + 2 \cdot H_{\text{barrier}}$  for the interior overhang region and  $L_{c\_end} + H_{\text{barrier}}$  for the end overhang region. Then:

$$F_{\text{cadj\_int}} = \frac{F_{\text{coll\_int}}}{L_{c\_int} + 2 \cdot H_{\text{barrier}}} = \frac{64.4}{10.9 + 2 \cdot \frac{38}{12}} = 3.7 \text{ kips/ft}$$

$$F_{\text{cadj\_end}} = \frac{F_{\text{coll\_end}}}{L_{c\_end} + H_{\text{barrier}}} = \frac{64.4}{5.0 + \frac{38}{12}} = 7.9 \text{ kips/ft}$$

The resulting load  $M_{\text{cadj}}$  is located at the top of the partial depth deck at the toe of the barrier. Translate this load to the center of the partial depth deck for design of the deck overhang. Referring to Figure 9.3.5, first find the eccentricity:

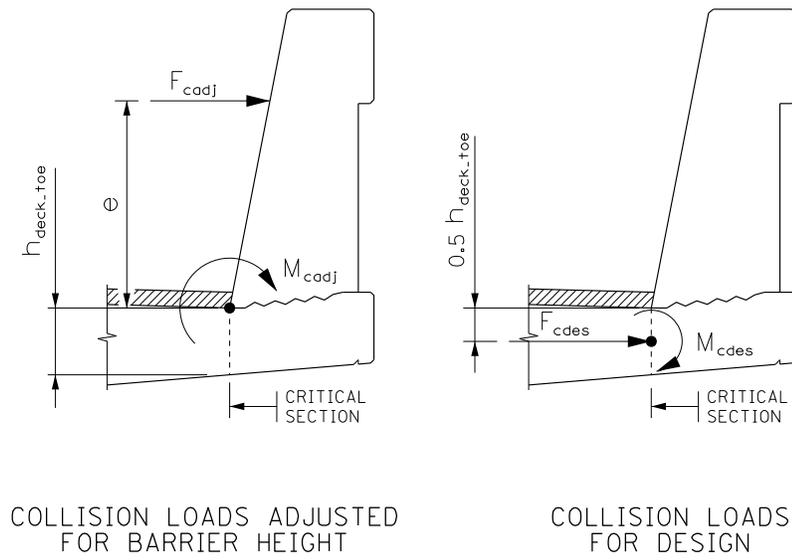


Figure 9.3.5

$$e_{int} = \frac{M_{cadj\_int}}{F_{cadj\_int}} = \frac{9.4}{3.7} = 2.54 \text{ ft}$$

$$e_{end} = \frac{M_{cadj\_end}}{F_{cadj\_end}} = \frac{20.5}{7.9} = 2.59 \text{ ft}$$

Then:

$$F_{cdes\_int} = F_{cadj\_int} = 3.7 \text{ kips/ft}$$

$$\begin{aligned} M_{cdes\_int} &= F_{cdes\_int} \cdot (e_{int} + 0.5 \cdot h_{deck\_toe}) \\ &= 3.7 \cdot \left( 2.54 + 0.5 \cdot \frac{8.84}{12} \right) = 10.8 \frac{\text{kip-ft}}{\text{ft}} \end{aligned}$$

$$F_{cdes\_end} = F_{cadj\_end} = 7.9 \text{ kips/ft}$$

$$\begin{aligned} M_{cdes\_end} &= F_{cdes\_end} \cdot (e_{end} + 0.5 \cdot h_{deck\_toe}) \\ &= 7.9 \cdot \left( 2.59 + 0.5 \cdot \frac{8.84}{12} \right) = 23.4 \frac{\text{kip-ft}}{\text{ft}} \end{aligned}$$

**Extreme Event II Limit State Bending Moment**

Total factored loads are:

**[A13.4.1]**

$$\begin{aligned} M_{u\_int} &= 1.00 \cdot M_{DC} + 1.00 \cdot M_{cdes\_int} \\ &= 1.00 \cdot 0.57 + 1.00 \cdot 10.8 = 11.4 \text{ kip-ft/ft} \end{aligned}$$

$$\begin{aligned} P_{u\_int} &= 1.00 \cdot F_{DC} + 1.00 \cdot F_{cdes\_int} \\ &= 1.00 \cdot 0.0 + 1.00 \cdot 3.7 = 3.7 \text{ kips/ft} \end{aligned}$$

$$\begin{aligned} M_{u\_end} &= 1.00 \cdot M_{DC} + 1.00 \cdot M_{cdes\_end} \\ &= 1.00 \cdot 0.57 + 1.00 \cdot 23.4 = 24.0 \text{ kip-ft/ft} \end{aligned}$$

$$\begin{aligned} P_{u\_end} &= 1.00 \cdot F_{DC} + 1.00 \cdot F_{cdes\_end} \\ &= 1.00 \cdot 0.0 + 1.00 \cdot 7.9 = 7.9 \text{ kips/ft} \end{aligned}$$

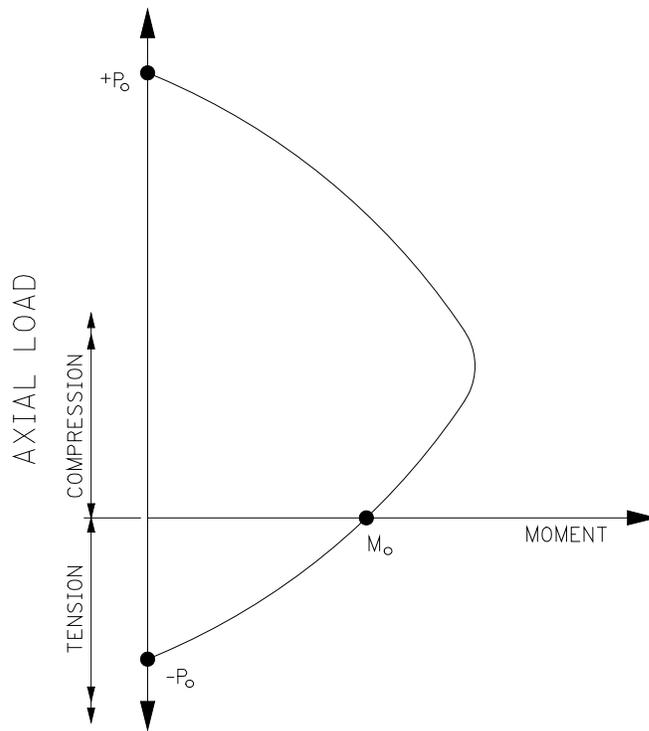
The eccentricity of  $P_u$  is:

$$e_{u\_int} = \frac{M_{u\_int}}{P_{u\_int}} = \frac{11.4}{3.7} = 3.08 \text{ ft} = 36.96 \text{ in}$$

$$e_{u\_end} = \frac{M_{u\_end}}{P_{u\_end}} = \frac{24.0}{7.9} = 3.04 \text{ ft} = 36.48 \text{ in}$$

**Resistance of Deck Interior Overhang Region**

The overhang must resist both axial tension and bending moment. The capacity of the overhang will be determined by considering the tension side of the structural interaction diagram for a one foot wide portion of the overhang. See Figure 9.3.6.



STRUCTURAL INTERACTION DIAGRAM

**Figure 9.3.6**

Check if the reinforcement chosen for the deck interior region (between the fascia beams) will be adequate for the overhang. The deck interior region reinforcement is:

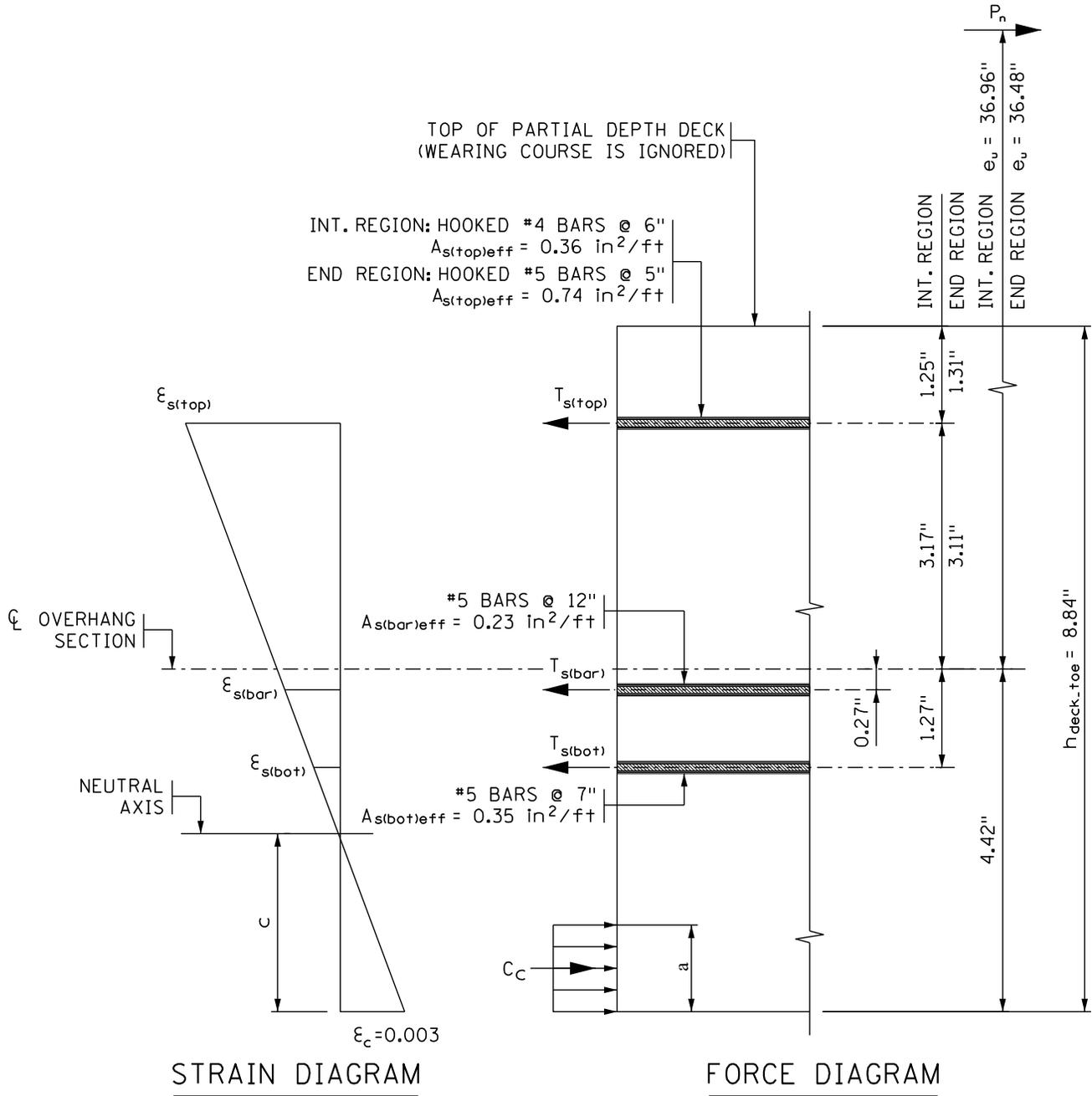
Top reinforcement - #4 bars @ 6" ( $A_{s(top)} = 0.40 \text{ in}^2/\text{ft}$ )

Bottom reinforcement - #5 bars @ 7" ( $A_{s(bot)} = 0.53 \text{ in}^2/\text{ft}$ )

Note that the front leg of the barrier bar also contributes to the strength of the overhang. The barrier reinforcement is:

Barrier front leg - #5 bars @ 12" ( $A_{s(bar)} = 0.31 \text{ in}^2/\text{ft}$ )

Referring to Figure 9.3.7, determine the capacity of the overhang section for the eccentricity  $e_{u,int}$  equal to 36.96 inches.



**Reinforced Concrete Section at Toe of Barrier**  
**Figure 9.3.7**

First, determine the effective area of reinforcement for the deck bars located at the toe of the barrier. From Figure 5.2.2.2 of this manual, the development length,  $l_d$ , for the deck bars are as follows:

For #4 top bars @ 6", cover is > 3" outside barrier toe, which results in an  $l_d = 18"$ .

For #5 bottom bars @ 7", cover varies with 1" minimum.

Using the 1" minimum cover results in an  $\ell_d = 33"$ .

For #5 barrier bars @ 12", minimum cover is 2.38" to the top of deck at the inside of the barrier toe (ignoring wearing course), which results in an  $\ell_d = 22"$ .

Then:

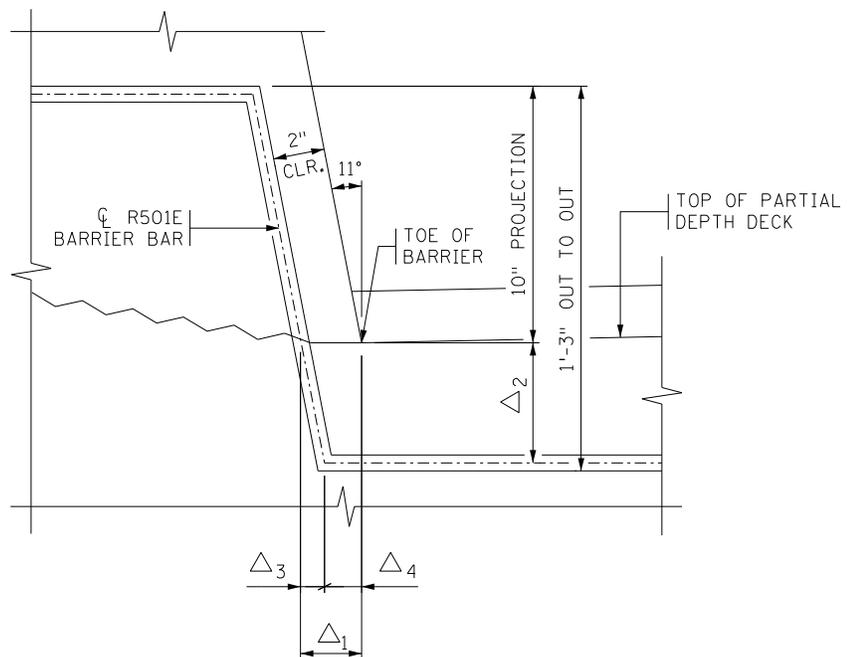
For #4 top bars,  $\ell_{d\text{available}} = L_{\text{crit}} - (\text{top bar end cover})$   
 $= 18.38 - 2.0 = 16.38 \text{ in}$

$$A_{s(\text{top})\text{eff}} = A_{s(\text{top})} \cdot \frac{\ell_{d\text{available}}}{\ell_d} = 0.40 \cdot \frac{16.38}{18} = 0.36 \frac{\text{in}^2}{\text{ft}}$$

For #5 bottom bars,  $\ell_{d\text{available}} = L_{\text{crit}} - (\text{bottom bar end cover})$   
 $= 18.38 - 4.0 = 14.38 \text{ in}$

$$A_{s(\text{bot})\text{eff}} = A_{s(\text{bot})} \cdot \frac{\ell_{d\text{available}}}{\ell_d} = 0.53 \cdot \frac{14.38}{33} = 0.23 \frac{\text{in}^2}{\text{ft}}$$

For #5 barrier bars (R501E), the bar is considered fully developed on the outside of the barrier toe due to the bend. On the traffic side of the barrier toe, refer to Figure 9.3.8 to determine how much of the 18" bar leg extends beyond the toe.



PARTIAL SECTION AT TOE OF BARRIER

Figure 9.3.8

$$\Delta_1 = \frac{2 + 0.5 \cdot (0.625)}{\cos 11^\circ} = 2.36 \text{ in}$$

$$\Delta_2 = 15 - 10 - 0.5 \cdot (0.625) = 4.69 \text{ in}$$

$$\Delta_3 = 4.69 \cdot (\tan 11^\circ) = 0.91 \text{ in}$$

$$\Delta_4 = 2.36 - 0.91 = 1.45 \text{ in}$$

Then:

$$\ell_{\text{davailable}} = 18 - 1.45 = 16.55 \text{ in}$$

$$A_{s(\text{bar})\text{eff}} = A_{s(\text{bar})} \cdot \frac{\ell_{\text{davailable}}}{\ell_d} = 0.31 \cdot \frac{16.55}{22} = 0.23 \frac{\text{in}^2}{\text{ft}}$$

Now determine the distance from the bottom of the section to the neutral axis,  $c$ . Start by assuming that for all reinforcement,  $\epsilon_s > \epsilon_y$ .

Then:

$$f_s = E_s \cdot \epsilon_y = f_y$$

$$T_{s(\text{top})} = A_{s(\text{top})\text{eff}} \cdot f_y = 0.36 \cdot 60 = 21.60 \text{ kips/ft}$$

$$T_{s(\text{bot})} = A_{s(\text{bot})\text{eff}} \cdot f_y = 0.23 \cdot 60 = 13.80 \text{ kips/ft}$$

$$T_{s(\text{bar})} = A_{s(\text{bar})\text{eff}} \cdot f_y = 0.23 \cdot 60 = 13.80 \text{ kips/ft}$$

$$T_{s(\text{tot})} = 21.60 + 21.00 + 13.80 = 49.20 \text{ kips/ft}$$

The total compression force  $C_c$  is:

$$C_c = 0.85 \cdot f'_c \cdot b \cdot a = 0.85 \cdot 4.0 \cdot 12.0 \cdot 0.85 \cdot c = 34.68 \cdot c$$

Referring to Figure 9.3.7, find  $c$  by taking moments about  $P_n$ :

$$\begin{aligned} & 21.60 \cdot (36.96 - 3.17) \\ & + 13.80 \cdot (36.96 + 1.27) \\ & + 13.80 \cdot (36.96 + 0.27) \\ & - 34.68 \cdot c \cdot (36.96 + 4.42 - 0.5 \cdot 0.85 \cdot c) = 0 \end{aligned}$$

Solving, we get  $c = 1.25 \text{ in}$

Check if original assumption was correct, that  $\epsilon_s > \epsilon_y$ :

$$\epsilon_y = \frac{f_y}{E_s} = \frac{60}{29,000} = 0.00207$$

$$\epsilon_{s(\text{top})} = (8.84 - 1.25 - 1.25) \cdot \left(\frac{0.003}{1.25}\right) = 0.01522 > 0.00207$$

$$\epsilon_{s(\text{bot})} = (4.42 - 1.27 - 1.25) \cdot \left(\frac{0.003}{1.25}\right) = 0.00456 > 0.00207$$

$$\epsilon_{s(\text{bar})} = (4.42 - 0.27 - 1.25) \cdot \left( \frac{0.003}{1.25} \right) = 0.00696 > 0.00207$$

Therefore the assumption was correct.

Then,

$$C_c = 34.68 \cdot c = 34.68 \cdot 1.25 = 43.35 \text{ kips/ft}$$

And,

$$\begin{aligned} P_n &= T_{s(\text{top})} + T_{s(\text{bot})} + T_{s(\text{bar})} - C_c \\ &= 21.60 + 13.80 + 13.80 - 43.35 = 5.85 \text{ kips/ft} \end{aligned}$$

**[1.3.2.1]**

The resistance factor  $\phi$  for Extreme Event II limit state is 1.0. Therefore,

$$\phi \cdot P_n = 1.0 \cdot 5.85 = 5.85 \text{ kips/ft} > 3.7 \text{ kips/ft} \quad \text{OK}$$

$$\phi \cdot M_n = \phi \cdot P_n \cdot e_u$$

$$= 1.0 \cdot 5.85 \cdot 36.96 \cdot \frac{1}{12} = 18.02 \frac{\text{kip-ft}}{\text{ft}} > 11.4 \frac{\text{kip-ft}}{\text{ft}} \quad \text{OK}$$

Therefore, the deck interior overhang region reinforcement is adequate.

**Resistance of Deck End Overhang Region**

The process for checking the end overhang region is the same as for the interior region, except the reinforcement differs. The Memo to Designers (2017-01) directs the designer to use Table 9.2.1.1 for the bottom reinforcement and modify the top reinforcement in regions near an expansion joint. For a 36 inch Type S barrier supported by prestressed beams, use the following reinforcement in accordance with the memo:

Top reinforcement – Hooked #5 bars @ 5" ( $A_{s(\text{top})} = 0.74 \text{ in}^2/\text{ft}$ ) over a distance of 8 feet from the joint.

Bottom reinforcement – #5 bars @ 7" ( $A_{s(\text{bot})} = 0.53 \text{ in}^2/\text{ft}$ )

In the end region, the front leg of the barrier bar also contributes to the strength of the overhang. There is some variation in bar spacing in the end region, but we will conservatively use:

Barrier front leg - #5 bars @ 12" ( $A_{s(\text{bar})} = 0.31 \text{ in}^2/\text{ft}$ )

First, determine the effective area of reinforcement for the deck bars located at the toe of the barrier. From Figure 5.2.2.6 of this manual, the development length,  $\ell_d$ , for the hooked top bars is:

For #5 top bars @ 5" with side cover > 2.5" and 2" end cover,  $\ell_d = 12"$   
 $\ell_{d\text{available}} = 16.38" > 12"$ , so top bars are fully developed.

$$A_{s(\text{top})\text{eff}} = 0.74 \text{ in}^2/\text{ft}$$

The other bars are unchanged, so:

$$A_{s(bot)eff} = 0.23 \text{ in}^2/\text{ft}$$

$$A_{s(bar)eff} = 0.23 \text{ in}^2/\text{ft}$$

Now determine the distance from the bottom of the section to the neutral axis,  $c$ . Start by assuming that for all reinforcement,  $\epsilon_s > \epsilon_y$ . Then:

$$f_s = E_s \cdot \epsilon_y = f_y$$

$$T_{s(top)} = A_{s(top)eff} \cdot f_y = 0.74 \cdot 60 = 44.40 \text{ kips/ft}$$

$$T_{s(bot)} = A_{s(bot)eff} \cdot f_y = 0.23 \cdot 60 = 13.80 \text{ kips/ft}$$

$$T_{s(bar)} = A_{s(bar)eff} \cdot f_y = 0.23 \cdot 60 = 13.80 \text{ kips/ft}$$

$$T_{s(tot)} = 44.40 + 13.80 + 13.80 = 72.00 \text{ kips/ft}$$

The total compression force  $C_c$  is:

$$C_c = 0.85 \cdot f'_c \cdot b \cdot a = 0.85 \cdot 4.0 \cdot 12.0 \cdot 0.85 \cdot c = 34.68 \cdot c$$

Referring to Figure 9.3.7, find  $c$  by taking moments about  $P_n$ :

$$44.40 \cdot (36.48 - 3.11)$$

$$+ 13.80 \cdot (36.48 + 1.27)$$

$$+ 13.80 \cdot (36.48 + 0.27)$$

$$- 34.68 \cdot c \cdot (36.48 + 4.42 - 0.5 \cdot 0.85 \cdot c) = 0$$

$$\text{Solving, we get } c = 1.80 \text{ in}$$

Check if original assumption was correct, that  $\epsilon_s > \epsilon_y$ :

$$\epsilon_y = \frac{f_y}{E_s} = \frac{60}{29,000} = 0.00207$$

$$\epsilon_{s(top)} = (8.84 - 1.31 - 1.80) \cdot \left(\frac{0.003}{1.80}\right) = 0.00955 > 0.00207$$

$$\epsilon_{s(bot)} = (4.42 - 1.27 - 1.80) \cdot \left(\frac{0.003}{1.80}\right) = 0.00225 > 0.00207$$

$$\epsilon_{s(bar)} = (4.42 - 0.27 - 1.80) \cdot \left(\frac{0.003}{1.80}\right) = 0.00392 > 0.00207$$

Therefore the assumption was correct.

Then the compression force  $C_c$  is:

$$C_c = 34.68 \cdot c = 34.68 \cdot 1.80 = 62.42 \text{ kips/ft}$$

And,

$$P_n = T_{s(top)} + T_{s(bot)} + T_{s(bar)} - C_c$$

$$= 44.40 + 13.80 + 13.80 - 62.42 = 9.58 \text{ kips/ft}$$

**[1.3.2.1]**

The resistance factor  $\phi$  for Extreme Event II limit state is 1.0. Therefore,

$$\phi \cdot P_n = 1.0 \cdot 9.58 = 9.58 \text{ kips/ft} > 7.9 \text{ kips/ft} \quad \text{OK}$$

$$\phi \cdot M_n = \phi \cdot P_n \cdot e_u$$

$$= 1.0 \cdot 9.58 \cdot 36.48 \cdot \frac{1}{12} = 29.12 \frac{\text{kip-ft}}{\text{ft}} > 24.0 \frac{\text{kip-ft}}{\text{ft}} \quad \text{OK}$$

Therefore, the end region deck reinforcement is adequate for the end overhang region under Case 1.

**L. Overhang  
Region Analysis,  
Case 2**

Case 2 is Extreme Event II checked at the edge of the beam flange for the dead load plus horizontal collision force plus live load. As noted earlier, live load is not considered due to the overhang geometry. At the edge of the beam flange, the analysis is very similar to that done for Case 1, except the dead load moment will be greater, the cross-section will be deeper, and the  $A_{s(\text{bar})\text{eff}}$  will be substantially less. Because the process is similar, the calculations are not shown and only the important values are included below to confirm the overhang is adequate.

Table 9.3.3 shows the loads for Case 2 and Table 9.3.4 shows results for the overhang resistance to the loads.

**Table 9.3.3 – Determination of Loads for Case 2 Applied to a 1 ft Wide Strip**

	Total W <sub>DC</sub> (kips)	Total M <sub>DC</sub> (kip-ft)	M <sub>cdj</sub> (kip-ft)	F <sub>cdj</sub> (kips) ①	e (ft)	F <sub>cdes</sub> (kips)	M <sub>cdes</sub> (kip-ft)	M <sub>u</sub> (kip-ft)	P <sub>u</sub> (kips)	e <sub>u</sub> (ft)
Interior region	0.763	0.970	9.4	3.5	2.7	3.5	10.8	11.8	3.5	3.37
End region	0.763	0.970	20.5	7.4	2.8	7.4	23.6	24.6	7.4	3.32

① Because section for analysis is at the beam flange, the collision load is distributed over:  
 $L_c + 2 \cdot H_{\text{barrier}} + 2 \cdot (\text{distance from edge of beam flange to barrier toe})$  for interior region.  
 $L_c + 1 \cdot H_{\text{barrier}} + 1 \cdot (\text{distance from edge of beam flange to barrier toe})$  for end region.  
 See Figure 9.3.9.

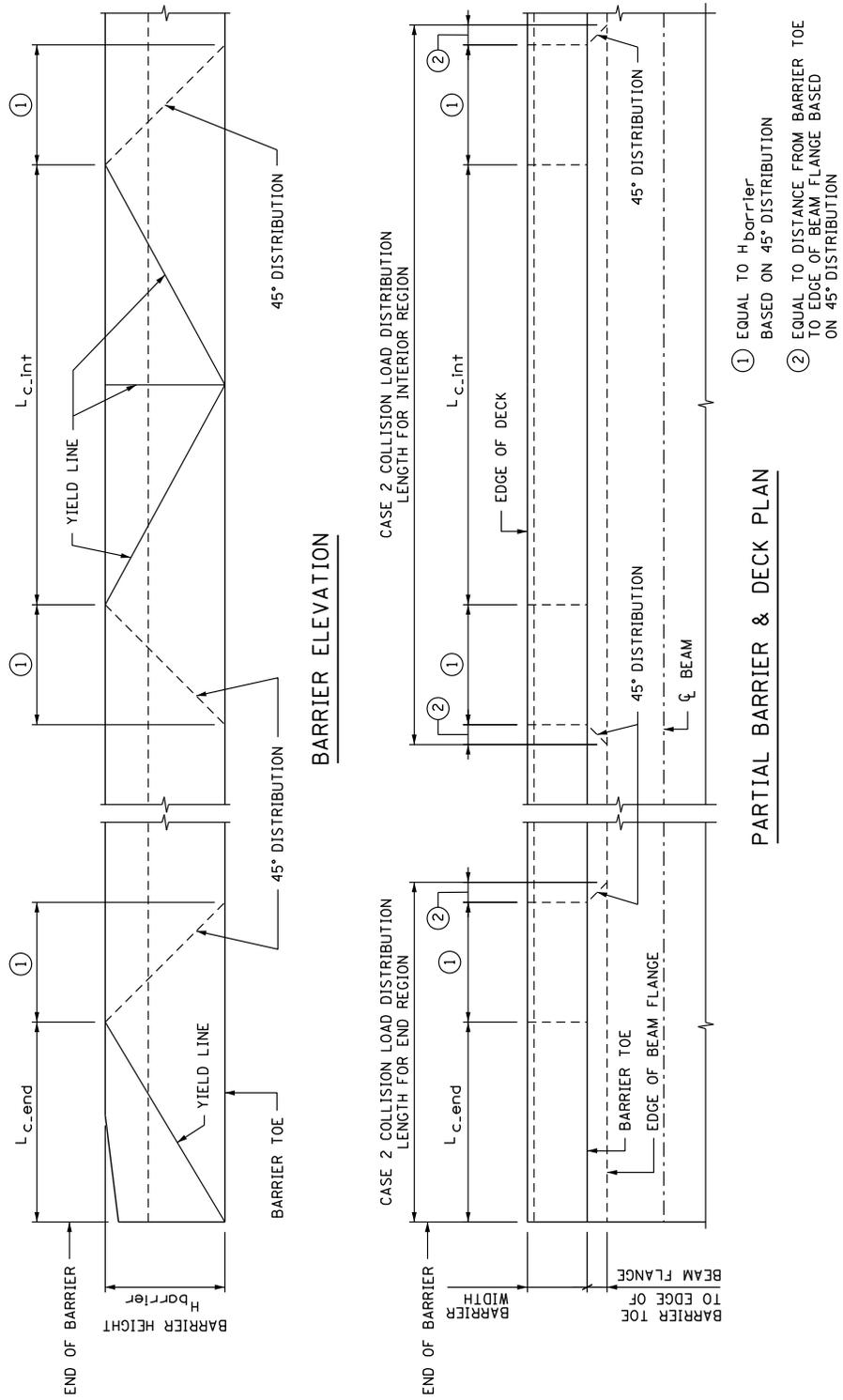


Figure 9.3.9

**Table 9.3.4 – Determination of Deck Overhang Resistance for a 1 ft Wide Strip**

	Overhang Reinforcement	Effective $A_s$ (in <sup>2</sup> )	Assumed $T_s$ (kips) ①	Arm to $\phi P_n$ (in)	$c$ (in)	Calculated $\epsilon_s > \epsilon_y$ ?	$C_{conc}$ (kips)	$\phi P_n$ (kips)	$\phi M_n$ (kip-ft)
Interior region	Top bars: #4 @ 6"	0.40	24.00	36.94	1.34	0.01547 > $\epsilon_y$ OK	46.47	6.3	21.33
	Bottom bars: #5 @ 7"	0.34	20.40	41.38		0.00553 > $\epsilon_y$ OK			
	Barrier bars: #5 @ 12"	0.14	8.40	40.38		0.00777 > $\epsilon_y$ OK			
End region	Top bars: hooked #5 @ 5"	0.74	44.40	36.40	1.83	0.01043 > $\epsilon_y$ OK	63.46	9.7	32.34
	Bottom bars: #5 @ 7"	0.34	20.40	40.78		0.00325 > $\epsilon_y$ OK			
	Barrier bars: #5 @ 12"	0.14	8.40	39.78		0.00489 > $\epsilon_y$ OK			

① Assumed  $T_s = (\text{Effective } A_s) \cdot f_y$

Based on the table values, the overhang deck reinforcement is adequate for Case 2.

**M. Overhang Region Analysis, Case 4**

The vertical collision load,  $F_v$ , for NCHRP Report 350 Test Level 4 represents the weight of a vehicle lying on top of the barrier and is applied over a length,  $L_v$ :

$$F_v = 18 \text{ kips}$$

$$L_v = 18 \text{ feet.}$$

The moment arm,  $L_{arm}$ , is:

$$L_{arm} = (\text{overhang}) - (\text{coping}) - (\text{barrier width}) - 0.5 \cdot b_f + x_{cg}$$

$$= 42 - 2 - 16.38 - 0.5 \cdot 34 + 10.22$$

$$= 16.84 \text{ in} = 1.40 \text{ ft}$$

Then, conservatively using a distribution length equal to  $L_v$ :

$$M_{v\text{collision}} = \frac{F_v}{L_v} \cdot L_{arm} = \frac{18}{18} \cdot 1.40 = 1.40 \frac{\text{kip-ft}}{\text{ft}}$$

Dead load is the same as calculated previously for Case 2:

$$M_{DC} = 0.970 \text{ kip-ft/ft}$$

Then  $M_u$  for Case 4 is:

$$\begin{aligned}\text{Case 4 } M_u &= M_{\text{vcollision}} + M_{\text{DC}} \\ &= 1.40 + 0.970 \\ &= 2.37 \text{ kip-ft/ft} \ll \text{Case 2 } M_u = 11.8 \text{ kip-ft/ft}\end{aligned}$$

Therefore, by inspection, Case 4 is satisfied.

The deck overhang reinforcement is adequate for all cases.

## **10. FOUNDATIONS**

Different types of foundations are used throughout the state due to the variety of soil and rock conditions present. This section provides guidance on the design and detailing practices for spread footings, driven piles, and drilled shaft foundations.

### **10.1 Determination of Foundation Type and Capacity**

During preliminary design a number of activities take place to determine the types of foundations to be used and the permitted capacities for foundation components.

Prior to beginning final design on trunk highway projects, designers should review the Foundation Engineer's Memo and the Bridge Construction Unit's Foundation Recommendations.

For bridges on the local road system, the local agency or their consultant will retain a private geotechnical engineering firm to prepare a foundation recommendations report. The report will summarize the geotechnical conditions, the proposed bridge structure, and recommend a foundation type.

#### **10.1.1 Foundation Engineer's Memo**

After conducting an exploration program, Mn/DOT's Foundation Engineer summarizes the geotechnical conditions at the site in a memo. The Regional Bridge Construction Engineer reviews the Foundation Engineer's Memo and the Preliminary Plans for the project and prepares the final recommendations concerning the foundations for the project. A sample Bridge Construction Unit Foundation Recommendation is provided in Appendix 10-A.

#### **10.1.2 Foundation Recommendations**

##### **Type and Size**

Based on geotechnical information and the anticipated type of structure, a foundation type will be recommended. In most cases pile supported footings will be recommended. The piling may be timber, cast-in-place concrete, H-pile, or pipe pile. Where scour is not a concern and soil or rock with adequate bearing capacity is found near the surface, spread footings may be recommended. Occasionally, a footing supported on drilled shafts will be recommended.

##### **Load Capacity**

The factored bearing resistance ( $\phi_b q_n$ ) for the material below spread footings and/or the factored bearing resistance ( $\phi R_n$ ) for piles or shafts will be provided in the Foundation Recommendations.

**Settlement/Downdrag**

The Foundation Recommendations often specify that an embankment be placed to allow settlement to occur before starting construction of a substructure. A waiting period of 72 hours to several months is then required depending on the types of underlying soils. In some cases, a surcharge embankment (additional height of fill above the profile grade) may also be recommended as a means of accelerating the rate of consolidation.

Depending on the soil profile and length of the settlement waiting period, long term settlement of the soil may introduce downdrag in the piling or shafts. Downdrag is the downward load induced in the pile by the settling soil as it grips the pile due to negative side friction. An estimate of the downdrag load will be given in the Foundation Engineer's Memo.

For piles driven to rock or a dense layer (where pile capacity is controlled by end bearing), the nominal pile resistance should be based on the structural capacity of the pile. For piles controlled by side friction, downdrag will apply a load to the pile that may cause pile settlement. The settlement may result in a reduction of the downdrag load. Due to the uncertainty of the amount of pile settlement, downdrag on friction piles will be considered on a case by case basis.

The amount of downdrag load to consider for design will be specified in the Foundation Recommendations. Note that Mn/DOT has not seen any bridge strength or serviceability problems that have been attributed to downdrag.

**[3.11.8]**

Transient loads have the effect of reducing downdrag. Therefore, when determining load combinations, do not combine live load (or other transient loads) with downdrag. Consider a load combination that includes dead load plus live load and also a load combination that includes dead load plus downdrag, but do not consider live load and downdrag within the same load combination.

Before using battered piles where downdrag loads exist, discuss with Bridge Design Engineer and Regional Bridge Construction Engineer.

**Method of Construction Control**

To ensure that foundations will have the capacities anticipated during design, testing or observations are made during construction. These construction controls consist of compaction testing for spread footings, the Mn/DOT Nominal Resistance Pile Driving Formula, Pile Driving Analyzer (PDA) testing, or physical load tests for piling and Cross-hole

Sonic Logging (CSL) for drilled shafts. The Foundation Recommendations will identify the construction controls to be used for the project.

#### **Estimated Pile Length**

The soil exploration program will not completely describe the geotechnical conditions at the site. To account for this variability, estimated pile lengths are used in computing bid quantities. Test pile lengths longer than anticipated production pile lengths (typically 10 feet longer) are specified in the Foundation Recommendations. If during construction, the test piles indicate that a longer or shorter length is justified, the production piling quantities and payments are adjusted accordingly.

#### **Estimated Bottom of Footing Elevation**

To minimize the potential for scour, settlement, or frost heave problems a recommended bottom of footing elevation will be presented for each substructure location in the Foundation Recommendations.

#### **Other General Information Needed for Plan Preparation**

Check pile layouts for interference with in-place utilities (including overhead power lines), drains and existing piles/foundations.

Unique projects may have limits placed on the amount of noise and vibration that can be generated during construction.

### **10.2 Piles**

Several types of piling are available (treated or untreated timber, steel H and thick wall pipe piles, and cast-in-place concrete piles). The Regional Bridge Construction Engineer may recommend that more than one type or size be used for a project.

Steel H-piles are steel H-shaped sections that are usually fitted with manufactured points and driven to a required nominal bearing resistance. H-piles are generally specified for soil conditions where very hard driving is anticipated, including driving to bedrock. In some cases, high strength, small diameter, thick-walled pipe are permitted as a substitute for H-piles. If permitted, this will be indicated in the Foundation Recommendations.

Cast-in-place (CIP) piles are steel pipe piles with a plate welded to the bottom that are driven to a required nominal bearing resistance or to an estimated tip elevation. After driving, the inside of the shell is filled with concrete. Reinforcement may be needed if the pile is subjected to tension or flexure. CIP piles are generally considered to be displacement

piles, and are generally used when it is anticipated that the pile tip will not encounter bedrock or very hard driving.

The pay item "Pile Tip Protection" refers to manufactured points for H-piling. The pay item "Pile Points" refers to manufactured points that are used to protect the shells of cast-in-place piles during driving operations. The Regional Bridge Engineer's recommendations will identify whether or not tips or points should be used.

Quantities to be included in a final plan set for structures supported on piling are: 1) length of piling delivered, 2) length of piling driven, 3) number and length of test piles, and 4) pile tip protection or pile points.

Standard Details B201 and B202 contain the standard splices for cast-in-place pile shells and H-piling.

Pile and drilled shaft foundation plans should be dimensioned from working points.

#### **Lateral Load Resistance**

A parametric study was conducted for CIP and H-piles modeled in a single layer of sand to determine simplified lateral load capacities to use for design. A pile lateral load computer program and the axial load/moment interaction equation in LRFD Article 6.9.2 (see below) were used for the study.

$$\frac{P_u}{\phi P_n} + \frac{8}{9} \left( \frac{M_u}{\phi M_n} \right) \leq 1.0$$

$P_u$  = factored axial load, determined by considering the driveability of each pile and choosing the maximum load that each pile can be driven to without damage based on past experience. For values greater than those shown in Table 10.2.1 below, a separate analysis is required.

$\phi P_n$  = factored axial resistance, calculated per LRFD Articles 6.9.4 and 6.9.5

$\phi M_n$  = factored bending resistance, calculated per LRFD Articles 6.12.2.2 and 6.12.2.3

$M_u$  = maximum factored moment

Values for  $P_u$ ,  $\phi P_n$ , and  $\phi M_n$  were determined and the interaction equation was solved for the maximum factored moment  $M_u$ . The maximum factored lateral load resistance  $\phi R_{nh}$  was determined by

applying an incrementally increasing lateral load to the computerized pile and soil model under the axial load  $P_u$  until the resulting moment was equal to  $M_u$ .

Table 10.2.1 shows the factored lateral load resistance  $\phi R_{nh}$  for different piles from the parametric study. The resistance values are based on soil properties for loose sand with an internal friction angle of  $30^\circ$  to  $32^\circ$ . The computer program built-in P-y curves using the "Reese sand" properties and relevant soil modulus,  $k$ , were also used. Soils with properties weaker than that of loose sand require a separate analysis. For the CIP piles, the 3 ksi concrete in the piles was included in the total EI for deflection determination, and also for calculation of the axial strength  $\Phi P_n$  in the piles. The pile cap was assumed fixed in rotation and free in translation.

**Table 10.2.1: Lateral Load Resistance of Piles**

Pile Type	$F_y$ (ksi)	Wall t (in.)	$P_u$ (tons)	$\Phi R_{nh}$ (kips)
12" CIP	45	1/4	100	24
12" CIP	45	5/16	125	24
12" CIP	45	3/8	150	24
12" CIP	45	1/2	200	24
16" CIP	45	1/4	135	28
16" CIP	45	5/16	170	40
16" CIP	45	3/8	205	40
16" CIP	45	1/2	270	40
HP 10x42	50	NA	110	24
HP 12x53	47.8 *	NA	140	32
HP 14x73	43.9 *	NA	190	40

\* Actual  $F_y = 50$  ksi. HP section does not meet b/t ratio for compactness. A reduced  $F_y$  was used in the analysis to meet requirements per LRFD Article 6.9.4.2.

#### **Pile Load Table Include in the Bridge Plan**

Standard practice for construction of pile foundations is to drive piling to refusal or to drive piling to the required nominal pile bearing resistance indicated in the plan. The nominal pile bearing resistance is monitored in the field using the Mn/DOT Nominal Resistance Pile Driving Formula or by using Pile Driving Analyzer (PDA) testing. Two tables are required in the plan when pile foundations are used. (See Appendix 2-H, Section F.) The first table is used to report the factored loads calculated during design of the pile layout. The second table is used to show the nominal resistance that the pile must be driven to in the field, depending on the field control method used.

Use the following values for  $\phi_{dyn}$  based on the field control method used:

- $\phi_{dyn} = 0.40$  for Mn/DOT Nominal Resistance Pile Driving Formula
- $\phi_{dyn} = 0.65$  for Pile Driving Analyzer

### Test Piles

Each bridge substructure utilizing a pile-type foundation will typically require one or two test piles. Separate the test piles (use a maximum spacing of about 40 feet) within a foundation unit to facilitate a more accurate assessment by the Field Engineer of the in-situ soil characteristics. The Foundation Recommendation prepared by the Regional Bridge Construction Engineer will specify the number of test piles for each substructure unit. For abutments with all battered piles, place a test pile in the front and in the back row. For pier footings, place test piles near the center of the pile group. If possible, use vertical (plumb) test piles. Number and locate test piles on the Bridge Survey Plan and Profile sheets.

Test piles are used to establish the length for the pier and abutment foundation piles. Based on the pile penetration (number of blows per foot at the end of driving), the size of the pile driving equipment, and the length of the pile being driven, the pile's nominal resistance can be estimated. The procedure used to determine the pile's nominal resistance is described in Bridge Special Provision SB2005-2452.2.

On some projects when specified, foundation test piles are evaluated with electronic equipment attached to the pile during the driving process. This equipment, called a Pile Driving Analyzer or PDA, provides more specific information concerning the nominal resistance of the pile. A pay item for pile analysis must be included in the plan when the PDA is performed by the contractor.

### Pile Redriving

Pile redriving is specified in the Foundation Recommendation when the soils are of a type that additional bearing capacity can be gained after the pile has set for 24 hours or more. For this situation, include an item for pile redriving to compensate the contractor for redriving the pile(s) after the required setup time.

#### [10.7.1.5]

### Clear Spacing and Minimum Concrete Cover

The minimum concrete cover for piles is 9 inches. To facilitate pile driving operations, the minimum center-to-center pile spacing is 2'-6" with 3'-0" minimum preferred.

It may be necessary to increase the plan dimensions of a footing or pile cap when using battered piles to provide the minimum concrete cover of 9 inches.

The standard embedment into a pier or high parapet abutment footing for a driven pile is one foot and should be dimensioned in the plans. Assume the piles are pinned supports.

The standard pile embedment for a low parapet abutment footing is 2'-4".

#### **Battered Piles**

The standard pile batter for pier footings is 6 vertical on 1 horizontal. For abutments, the standard batter is 4 vertical on 1 horizontal. Use of a nonstandard batter requires approval from the Regional Bridge Construction Engineer.

Pile layouts for foundations that include battered piles should be dimensioned at the bottom of the footing.

Before using battered piles where downdrag loads exist, discuss with Bridge Design Engineer and Regional Bridge Construction Engineer.

### **10.3 Drilled Shafts**

Drilled shafts are large-diameter reinforced concrete piles constructed by boring a hole into earth and/or rock, inserting a reinforcing cage and filling the cavity with concrete. Drilled shafts may also be called caissons or drilled piers. Because of the high cost of construction, drilled shafts are normally used only when the foundation characteristics of the site, such as bedrock, may cause driven piling to attain bearing capacity at ten feet or less below the footing, when piling cannot be embedded below the computed scour elevation of a streambed, and for other reasons applicable to a particular project. Drilled shafts may also be used to enhance the stability of piers adjacent to a navigation channel.

Information used for the design of drilled shafts is determined by the Mn/DOT Foundations Unit. This information includes depth (length) of the earth and rock portions of the shaft, and maximum load capacity for a given diameter. Load capacity of drilled shafts is provided by end bearing on rock (minimum embedment five feet), or by sidewall friction in soil or rock.

Drilled shafts are designed as columns subjected to axial and lateral loads. Lateral loads may or may not be resisted by passive soil pressure,

i.e., scour depth below the streambed (flowline) should not be considered as providing lateral support. If shafts are placed in a group, the minimum center-to-center spacing is three times the diameter ( $D$ ) of the shaft and appropriate group reduction factors must be applied. When the spacing is greater than  $8D$  the shafts can be designed as individual units. Shaft diameter is determined by the required loading, standard industry drilling equipment, casing size, and other factors unique to the project. Normally, shaft diameters are in the range of 3 to 5 feet. Smaller shafts may be used to replace driven piles in a group, such as that of a pier footing. Larger shafts may be appropriate when a single shaft is used to support a single pier column, or to minimize the number of shafts in a group when deep shafts are required. For a combined earth and rock shaft, the earth portion should be of a diameter that is 6 inches larger than the rock shaft in order to allow passage of special rock drilling tools. If a shaft terminates in rock, the design diameter for the full depth of the shaft should be the same diameter as that of the rock portion.

Detailing of drilled shafts in the plans should consider location, construction methods, foundation conditions, contract administration by district construction personnel, structural integrity of finished shafts, etc. Many details are job specific; therefore, much of this information should be compiled before detailing is started.

Because most of the depth of a shaft is formed by the excavated borehole, it will be necessary to determine if casings, either permanent or temporary, will be used. Permanent casings must be specified whenever shafts are constructed in water, even if the work is contained within a cofferdam and the final cut off elevation is below the streambed because dewatering cannot take place before the shafts are constructed. Some contractors prefer that permanent casings be used through all soil to the top of bedrock in case any of the soil is capable of caving. Permanent casings should not be used in the sidewall friction area of soil or rock. Temporary casings are provided by the contractor for the convenience of construction operations and are removed at the completion of the work. Most casings are provided in diameters of 6 inch increments and should be specified as such. For metric plans, the diameter must be soft-converted to metric units and not rounded off. Otherwise, the contractor may provide custom-made casings at a higher price.

Drilled shafts are reinforced in the same manner as round columns. Cover on the bars should be 3 inches on the sides and 6 inches from the bottom of the shaft. If the shaft design requires a reinforced connection between the top of the shaft and the structure above and hooked bars are intended, the hooks projecting beyond the side of the shaft may

prevent subsequent removal of temporary casings. Hooks may be turned inward to avoid this interference; however, possible interference with placement of footing reinforcement should then be checked.

Uncoated reinforcement bars should be used unless the top portion of a shaft will be permanently exposed, or if the bars will be extended into an exposed portion of the structure. In this case, use coated bars only at the top of the shaft unless it is more practical to use coated bars throughout.

When specifying concrete for the shafts, the mix normally used is 1X46 ( $f'_c = 5.0$  ksi) if the concrete will be placed in a wet (water-filled) hole, and 1Y46 ( $f'_c = 4.0$  ksi) if the concrete will be placed in a dry hole. The first digit should be "3" for air-entrained concrete if the top portion of the shaft will be exposed in the final construction. Aggregate should be no larger than  $\frac{3}{4}$ " to provide for a positive flow around the reinforcement since vibration of the concrete in the greater part of the shaft is not practical.

Payment for the drilled shafts should always include separate items for earth and rock shafts due to the large disparity in the cost of drilling. If it appears to be unlikely that the shaft depth will change during construction, payment for concrete, reinforcement, and permanent casings (if used) can be included in the pay item for the shafts. However, foundation conditions are rarely known with a high degree of accuracy and changes in the shaft quantities may occur. For such situations, separate items for the materials are recommended. In either case, the plans and special provisions must clearly state how payment will be made.

When boulders can be anticipated during drilling, include a pay item for obstruction removal.

Because it is not possible to visually inspect the unexposed portion of a finished shaft, other means of inspection and structural integrity testing have been devised. One such test is Cross-hole Sonic Logging (CSL). This test and other tests should be used only if recommended by the Regional Bridge Construction Engineer since these tests and the preparation of the shafts for the tests can be very costly.

Report maximum factored design load and factored bearing resistance in the plans using one of the Standard Plan Note tables shown in Appendix 2-H, Section F.

**10.4 Footings****10.4.1 General**

Any footings or foundations with a thickness of 5 feet or greater should be treated as mass concrete. This may require the contractor to modify the concrete mix and/or to instrument the concrete member and take action to ensure that the temperature differential between the inside and outside of the member is small enough to minimize the potential for cracking.

**Minimum Soil Cover**

The minimum cover (soil, earth, or slope paving) on top of a footing is 12 inches. For a pier footing which extends under a roadway, the minimum cover is 2 feet.

**Bottom of Footing**

To minimize the potential for frost movements impacting the structure, place the bottom of footings a minimum of 4'-6" below grade. Note that this requirement does not apply to the bottom of integral abutment pile caps.

When feasible, the bottom of footings (or seals if they are used) should be placed below the estimated scour elevation. In many cases this is not economically practical and the bottom of footing elevation should be evaluated using Section 10.6 as a minimum criteria.

**Scour**

The scour depth to be used for the strength and service limit states is the lesser of the overtopping or 100-year flood.

The scour depth to be used for the extreme event limit state is the lesser of the overtopping or 500-year flood.

For bridges over a river or stream, spread footings are not allowed due to the potential for scour unless they are anchored into rock.

When designing footings in areas of potential scour assume no beneficial ground support for the piling or drilled shafts from the flowline to the predicted total scour elevation during the extreme event load case.

**Footing Plan Dimensions/Formwork**

Footing plan dimensions should be laid out in a manner that will allow support of the formwork used to construct the substructure elements above it. This is accomplished by extending the footing at least 6 inches beyond the vertical face of the wall or stem.

**Footing Thickness/Shear**

The footing thickness should be sized such that shear reinforcement is not required. Use the simplified shear method of LRFD 5.8.3.4.1 when the requirement for zero shear within  $3d_v$  from column/wall face is met. Otherwise, use the general procedure given in LRFD 5.8.3.4.2.

**Footing Flexure Steel and "d" dimensions**

For footings with a pile embedment of one foot or less, place flexural reinforcement on top of the cut off piles. For pile footings with an embedment greater than one foot, place reinforcement between the piles.

**Dowel Detailing**

Dowels connecting the footing to the substructure unit shall be detailed and dimensioned from working points. This reduces the chance of construction tolerances for pile driving and concrete placement impacting the final location of substructure components.

**10.4.2 Footings  
Supported on Piling  
or Drilled Shafts**

Dimension length of pile embedment into the footing in the plans. Identify battered piles with a symbol that differs from vertically driven piles.

**Seal Design**

A conventional cast-in-place seal is a mass of unreinforced concrete poured under water inside the sheet piling of a cofferdam. Refer to Figure 10.4.2.1. It is designed to withstand the hydrostatic pressure produced at the bottom of the seal when the water above is removed. Dewatering the cofferdam allows cutting of piles, placement of reinforcing steel, and pouring of the footing in a dry environment.

Design of the seal consists of determining a concrete thickness that will counterbalance the hydrostatic pressure with an adequate factor of safety. Design is done under the service limit state.

Lateral forces from stream flow pressure are resisted by penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. The cofferdam design is the responsibility of the contractor.

Use the following procedure for seal design.

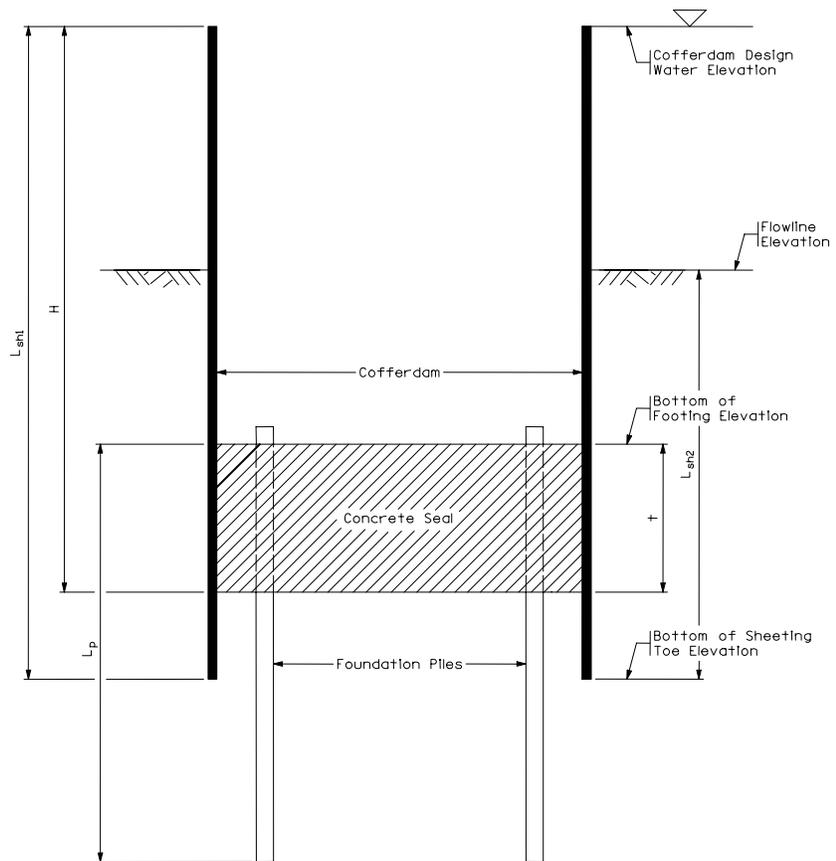


Figure 10.4.2.1

- 1) Determine preliminary dimensions:  
 A rule of thumb for preliminary seal thickness is  $0.25H$  for pile footings. The minimum allowed seal thickness is 3 feet. In plan, the minimum length and width of the seal is 1.5 feet larger than the footing on all sides, but it must also be large enough to avoid interference between sheet piling and battered piles.
  
- 2) Determine hydrostatic buoyancy force,  $P_b$ , due to hydrostatic pressure developed at the bottom of the seal:

$$P_b = H \cdot A \cdot \gamma_w$$

where  $H$  = hydrostatic head, ft

$A$  = plan area of cofferdam minus area of piles,  $\text{ft}^2$

$\gamma_w$  = unit weight of water,  $0.0624 \text{ kips/ft}^3$

- 3) Determine resistance due to seal weight,  $R_{sc}$  :

$$R_{sc} = A \cdot t \cdot \gamma_c$$

where  $t$  = thickness of seal, ft

$$\gamma_c = \text{unit weight of concrete, } 0.150 \text{ kips/ft}^3$$

- 4) Determine sheet piling resistance  $R_{sh}$ . This will be the smaller of:  
sheet piling weight  $P_{sh}$  + soil friction on sheet pile  $P_{shsoil}$   
or  
bond between sheet piling and seal  $P_{shseal}$

$$P_{sh} = L_{sh1} \cdot p_{cof} \cdot \omega_{sh}$$

where  $L_{sh1}$  = length of sheet piling in feet, normally based on sheet piling embedment of approximately  $H/3$

$$p_{cof} = \text{nominal perimeter of cofferdam, ft}$$

$$\omega_{sh} = \text{weight per square foot of sheet piling, normally assume } 0.022 \text{ kips/ft}^2$$

$$P_{shsoil} = L_{sh2} \cdot p_{cof} \cdot f_{shsoil}$$

where  $L_{sh2}$  = length of sheet piling below flowline in feet, normally based on sheet piling embedment of approximately  $H/3$  (choose conservative value for flowline elevation to account for scour or reduce by 5 ft)

$$f_{shsoil} = \text{friction of sheet piling with soil, normally assume } 0.150 \text{ kips/ft}^2$$

$$P_{shseal} = (t - 2) \cdot p_{cof} \cdot f_{shseal}$$

where  $f_{shseal}$  = bond of sheet piling to soil, normally assume  $1.0 \text{ kips/ft}^2$

- 5) Determine foundation piling resistance  $R_{pile}$ . This will be the smaller of:

foundation pile weight  $P_p$  + piling pullout resistance  $P_{ppull}$

or

the bond between foundation piling and seal  $P_{pileseal}$

The piling pullout resistance  $P_{ppull}$  is the smaller of:

soil friction on all individual piles  $P_{pilesoil}$

or

soil friction on pile group  $P_{grp}$  + weight of soil in pile group  $P_{soil}$

$$P_p = N \cdot [\omega_p \cdot L_p - (H + L_p - t) \cdot \gamma_w \cdot A_p]$$

where  $N$  = number of piles

$\omega_p$  = non-buoyant weight per foot of an unfilled pile, kips/ft

$L_p$  = estimated pile length

$A_p$  = end bearing area of pile, sq ft

$$P_{pilesoil} = N \cdot A_{psurf} \cdot f_{pilesoil} \cdot (L_p - t)$$

where  $A_{psurf}$  = surface area of pile per unit length, ft<sup>2</sup>  
(for H-piles, take  $A_{psurf} = 2 \cdot (\text{depth} + \text{width})$ )

$f_{pilesoil}$  = friction between piles and soil, normally assume  
0.150 kips/ft

$$P_{grp} = (L_p - t) \cdot f_{pilesoil} \cdot p_{grp}$$

where  $p_{grp}$  = perimeter of pile group, ft

$$P_{soil} = (L_p - t) \cdot A_s \cdot \gamma_{sb}$$

where  $A_s$  = area of soil engaged by pile group, which is the group perimeter area defined by the outside piles minus the area of the piles, ft<sup>2</sup> (use perimeter at top of pile group)

$\gamma_{sb}$  = buoyant unit weight of soil, 0.040 kips/ft<sup>3</sup>

$$P_{\text{pileseal}} = t \cdot N \cdot A_{\text{psurf}} \cdot f_{\text{pileseal}}$$

where  $A_{\text{psurf}}$  = surface area of pile per unit length,  $\text{ft}^2$   
(for H-piles, take  $A_{\text{psurf}} = 2 \cdot \text{depth} \cdot \text{width}$ )

$f_{\text{pileseal}}$  = friction between piles and seal, normally assume  
1.0 kips/ $\text{ft}^2$

- 6) Determine factor of safety, FS, and revise design as needed. The minimum required factor of safety is 1.2:

$$\text{FS} = (R_{\text{sc}} + R_{\text{sh}} + R_{\text{pile}}) / P_b$$

### **10.4.3 Spread Footings**

Abutment spread footings supported on rock shall be keyed into rock a minimum of 6 inches. Shear keys should be added to spread footings when needed. Typical shear keys are 12" x 12" or 18" x 18".

To ensure proper bearing capacity below spread footings founded on rock with variable elevation, a 1C63 concrete fill may be placed on the rock to provide a level foundation. Refer to the Foundation Recommendations.

To ensure proper bearing capacity below spread footings not founded on rock, a layer of aggregate with 100% compaction may be specified under spread footings. Refer to the Foundation Recommendations.

Report maximum factored design load and factored bearing resistance in the plans using one of the Standard Plan Note tables shown in Appendix 2-H, Section F.

### **10.5 Pile Bent Piers and Integral Abutments**

For pile bent piers, the pile tips should be driven a minimum of 10 feet below the scour elevation. The resistance of the piling needs to be checked for the condition where the predicted scour event has occurred. When debris loading can be excessive, encasing the piles with a concrete wall will be specified.

For integral abutments, orient H-piles for weak axis bending in the direction of movement and inform the Road Design group of the appropriate approach panel detail to include in the roadway plans.

For pile bent piers, provide 2'-0" of embedment into the cap. A larger pile embedment equal to 2'-6" is used for integral abutments.

**10.6 Evaluation of Existing Pile Foundations when Exposed by Scour**

The following guidelines may be used with discretion by registered engineers for determination of the stability of existing bridge substructure units supported by pile foundations (see Figure 10.6.1) if estimated scour depths are sufficient to expose piling. Estimated scour depths to be used are those furnished by the Hydraulics Engineer for the lesser of overtopping or a 500-year flood event.

- 1) For pile bent piers or abutments and for piers or abutments on footings supported by friction piling, the substructure unit is classified as stable with respect to scour if scour depth will not expose more than 50% of the embedded piling, and the unsupported pile length is not more than 24 times the diameter of cast-in-place pile, 24 times the nominal section depth of an H-pile, or 16 times the average diameter of a timber pile.
- 2) For pile bent piers or abutments or for piers or abutments on footings supported by end bearing piling, the substructure unit is classified as stable with respect to scour if at least 5 feet of the pile will remain embedded in dense material and the unsupported pile length meets the criteria in 1) above.

The substructure unit shall be considered stable if the foundation satisfies one of the above criteria. These guidelines are based on the concept that countermeasures will be taken where inspection reveals scour holes in the vicinity of pile bents or below the bottom of concrete footings. Pile exposures without lateral support will therefore be of relatively short duration.

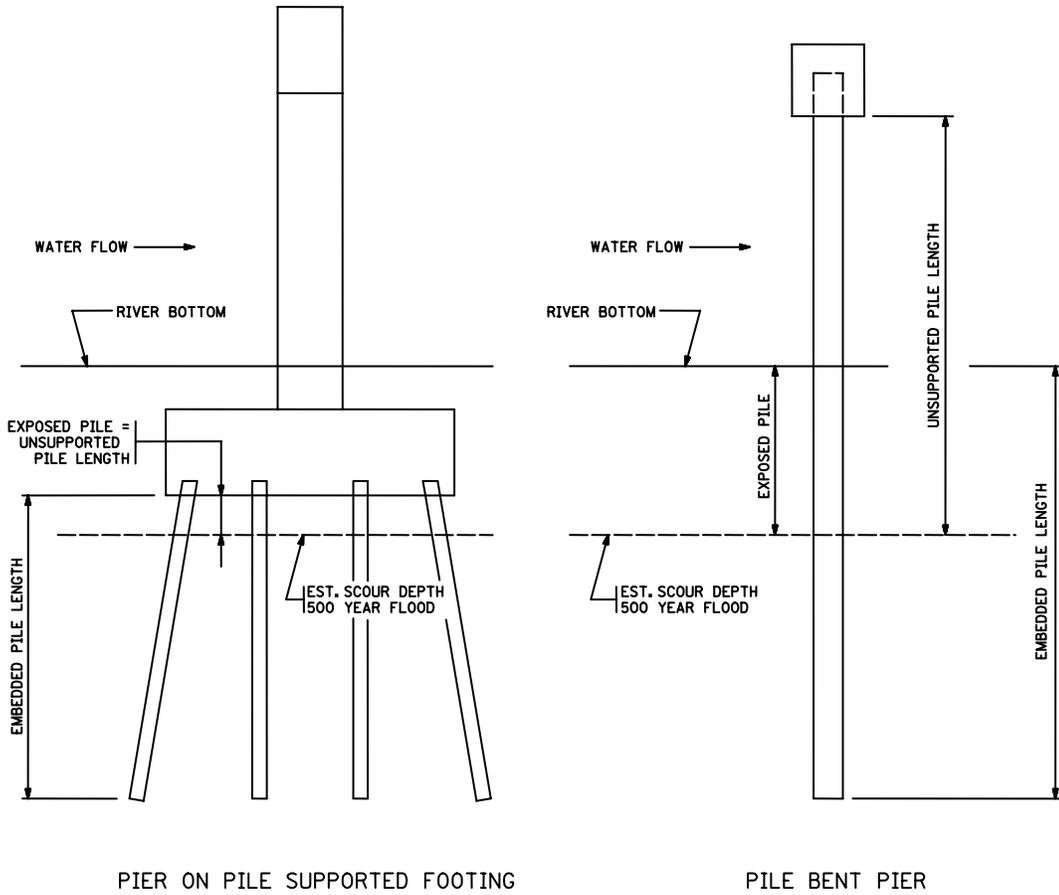


Figure 10.6.1

**10.7 Structure Excavation and Backfill**

For state aid projects, bridge designers must coordinate their excavation and fill quantities with roadway designers. This is particularly true for projects where grading is let as part of a separate contract. Designers should note the limits of excavation and fill noted in the standard bridge approach treatments (Mn/DOT Standard Plans 5-297.233 and 5-297.234).

The cost associated with excavating material, in and around foundations, depends on several items. These items include: access to the site, the amount of material that needs to be removed, the type of material to be removed (sand, silt, clay, rock, etc.), and the location of the water table. Mn/DOT's Spec. 2451 identifies and describes the standard classes (U, E, R, WE, WR) of excavation by the cubic yard.

Where no rock is present, use a lump sum pay item for structure excavation. The special provisions detail the percentage of excavation paid for at each substructure unit. Where rock is likely to be

encountered, pay for the rock excavation as Class R (or WR for rock below water) by the cubic yard. Excavation above the rock is to be paid for as a lump sum. Refer to the Foundation Recommendations.

When aggregate backfill is used under spread footings, the additional excavation below the bottom of footing elevation is considered incidental to placing the backfill material.

Class R excavation may be used by itself, in which case it would cover all conditions of rock removal. When used in conjunction with WR, the lower limits of the Class R should be noted in the Plans as being the same as the upper limits of the WR (the lower water elevation shown in the Plans). Because rock excavation is expensive, adequate boring or sounding information is essential to determine the elevation of the rock surface. If the information furnished is insufficient to determine the elevation of rock, additional data shall be requested from the District.

Appendix 10-A

Sample Bridge Construction Unit Foundation Recommendations

FOUNDATION AND OTHER RECOMMENDATIONS Bridge Construction Unit		Report No. <u>1630</u> Bridge No. <u>14216</u>
Substructure Unit	Approx. Station Along NB 34 <sup>th</sup> St	Location <u>34<sup>th</sup> St over I-94 (Maxhead)</u>
Estimated Bottom Elevation of Footing or Bent Cap	*Factored Spread Footing Bearing Resistance $\Phi_{q_n}$ (tsf)	*Factored Pile Bearing Resistance, $\Phi R_n$ (tons)
Sp. Abut.	125+6.7	100'-12" / 135'-16"
Pier 1	127+0.3	"
Pier 2	128+3.0	"
N.O. Abut.	129+2.5	"
*Based on Foundation Engineer's Recommendations dated _____		Scour Recommendations <u>N.A.</u>

Pile Type and Size	Estimated Pile Lengths	
Other	Steel H	C.I.P. Concrete
No.	Length	Foundation Pile Length
2	120'	110'
2	110'	95'
2	120'	105'
2	120'	110'

Remarks (Basis for above determinations):  
 The designer may use either 12"  $\phi$  0.25" wall thickness at  $\Phi R_n = 100$  tons, or 16"  $\phi$  0.25" wall thickness at  $\Phi R_n = 135$  tons.  
 When considering boundary loads, the designer may use the full structural capacity of the pile (204 tons for 12" CIP, or 310 tons for 16" CIP for  $\Phi R_n$ ).  
 The designer shall try to minimize the pile batter. See the Foundations Engineers Memo regarding settlements - as much as 30" long term. Include special provision language to require the use of the PDA to determine field pile resistance for all test piles.

Use special pay items for piling

Use thick wall pipe pile option

Use the following pile tip protection

Pile points \_\_\_\_\_

Pile tip protection \_\_\_\_\_

Use lump sum excavation item (except where rock excavation indicated)

Excavation to be incidental (to 1A43 Concrete or other)

Concrete seal required

\*Time delay recommended for approach embankment settlement: \_\_\_\_\_ months

72 Hrs.

None

Other Recommendations:

Use special concrete placement procedures on deck pours (for skewed bridges, etc.)

Paint color of exposed pile shells \_\_\_\_\_

Date preliminary received by reviewer <u>12/6/06</u>	Reviewed by <u>PAJ</u>	Concurred by <u>BAL</u> Date <u>12-13-06</u>
cc: Foundations Engineer, Preliminary Plans Engineer (3 copies), & Program Clerk		

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## **11. ABUTMENTS, PIERS, AND WALLS**

This section contains guidance for the design and detailing of abutments, piers, retaining walls, and noise walls. Abutments and piers are used to support bridge superstructures, whereas walls primarily function as earth retaining structures. In most cases, abutments, piers, and walls are reinforced concrete elements.

The preferred details for connecting the superstructure to the substructure are dependent on the geometry and type of bridge. For example, flexible substructure units supported by a single line of piles may be constructed integral with the superstructure. Conversely, stiff substructure units are detailed with expansion bearings between the superstructure and substructure to reduce the design loads in the substructure units.

### **11.1 Abutments**

#### **General**

Abutments function as both earth retaining structures and as vertical load carrying components. Integral and semi-integral abutments are designed to accommodate movements at the roadway end of the approach panel. Parapet abutments are detailed to accommodate movements with strip seal or modular expansion joint devices between the concrete deck and the abutment end block.

Railroad bridge abutments shall be designed according to the *AREMA Manual for Railway Engineering, Volume 2*, for the live load specified by the railroad. Design all other abutments according to the *AASHTO LRFD Bridge Design Specifications*. The Duluth Mesabe & Iron Range Railway requires a special live load. The live load surcharge is found by taking the axle load and distributing it over an area equal to axle spacing multiplied by the track spacing, generally 70 square feet. Do not reduce the surcharge loading for skew.

Refer to Article 2.4.1.6.2 when locating utilities near an abutment. When footings are perched on an embankment, consult with the Regional Construction Engineer regarding the use of spread footings.

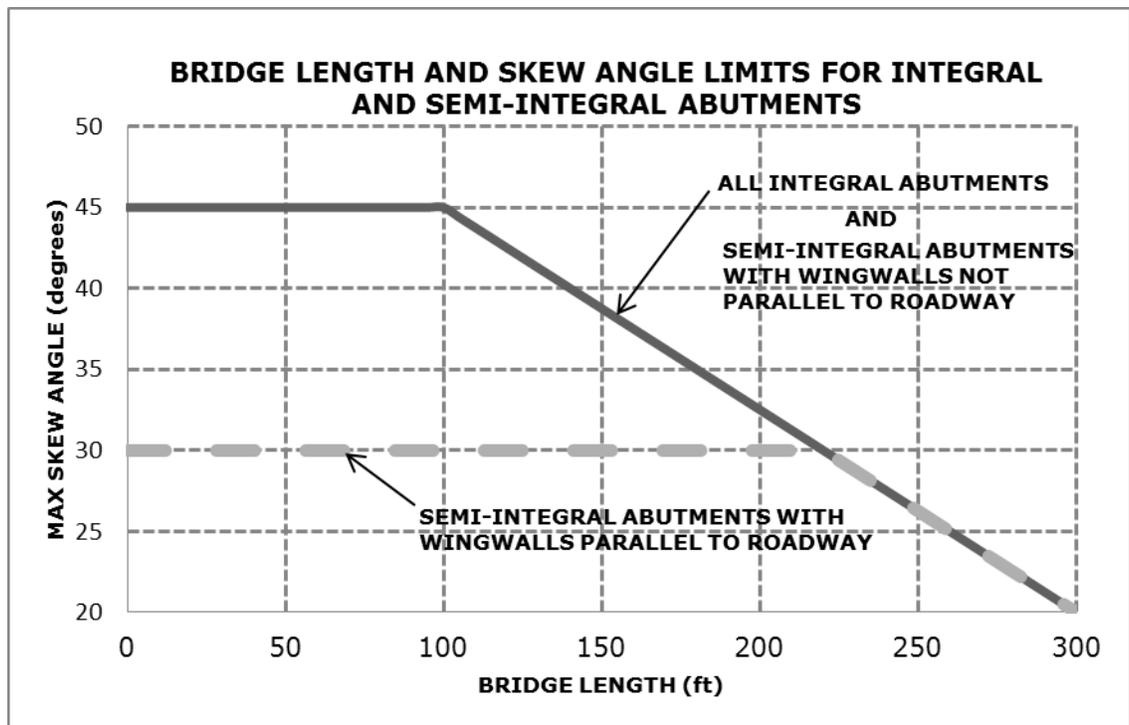
#### **Abutment Type Selection**

Integral abutments are the preferred type of abutment when all of the following criteria are met:

- The bridge length and skew meet one of the following:  
(See Figure 11.1.1)
  - Bridge length  $\leq$  300 feet and skew  $\leq$  20 degrees
  - Bridge length  $\leq$  100 feet and skew  $\leq$  45 degrees
  - Bridge length is between 100 feet and 300 feet, and

skew  $\leq [45 - 0.125 (L - 100)]$  degrees, where L is the length of the bridge in feet.

- Bridge horizontal alignment is straight. Slight curvature can be allowed, but must be considered on a case-by-case basis.
- The length of wingwall cantilevers are  $\leq 14$  feet (measured from the back face of abutment to the end of the wingwall).
- Abutment wingwalls do not tie into roadway retaining walls.
- Bridge configuration allows setting the abutment front face exposure on the low side of the bridge at 2 feet.
- Maximum abutment stem height  $\leq 7'-0''$
- Depth of beams is  $\leq 72$  inches.



**Figure 11.1.1**

Semi-integral abutments are the preferred type of abutment when the following circumstances apply:

- The wingwall length, abutment exposure or superstructure depth requirements for integral abutments cannot be met.
- The bridge length and skew meet the requirements given above for integral abutments, except that when wingwalls are parallel to the roadway, the maximum skew limit for semi-integral abutments is 30 degrees. (See Figure 11.1.1.) Also, note that a guide lug is required for skews greater than 30 degrees to limit unwanted lateral movement.

Parapet abutments should only be considered where integral and semi-integral abutment criteria cannot be met.

A parapet abutment supported by a pile foundation can be used behind a mechanically stabilized earth (MSE) retaining wall where high abutments would be required and where it is economical to use an MSE wall. Locate the front face of the MSE wall a minimum of 6'-0" from the centerline of bearing. Do not batter the piles. Place the bottom of the abutment footing and the bottom of the MSE wall cap at the same elevation. Slope protection between the abutment and the MSE wall cap should not exceed a 1V:4H slope.

### **Detailing/Reinforcement**

For bridge rail sections that extend beyond the bridge ends and connect to guardrail, it is preferable to locate them on top of the approach panel rather than on top of the wingwall. However, for situations where the wingwalls tie into roadway retaining walls, be aware that this will result in an offset between the wingwall and roadway retaining wall. In this case, additional coordination with the roadway designer will be required.

Extend architectural rustications 2 feet below the top of finished ground.

As a minimum, tie abutment and wingwall dimensions to the working points by providing distances normal and parallel to the center line of bearing from working points to the following points:

- Centerline of piles at abutment footing corners
- Corners of abutment front face
- Corners of abutment fillets
- Wingwall ends at front and back face of wall

The gutter line, the edge of deck, and the centerline of the fascia beam should be illustrated and labeled in the corner details.

To facilitate plan reading, label the ends of the abutments in the details (South End, North End, etc.).

Label all construction joints and identify the nominal size of keyways.

Where conduit extends through an abutment, provide horizontal dimensions from a working point to the location where the conduit penetrates the front face of the abutment or the outside face of the wingwall. The elevation at mid-height of the conduit should also be provided.

For presentation clarity, detail abutments with complicated layouts on separate sheets. Identical abutments (except for minor elevation differences) should be detailed on common sheets.

The minimum depth for the paving bracket is 1'-4".

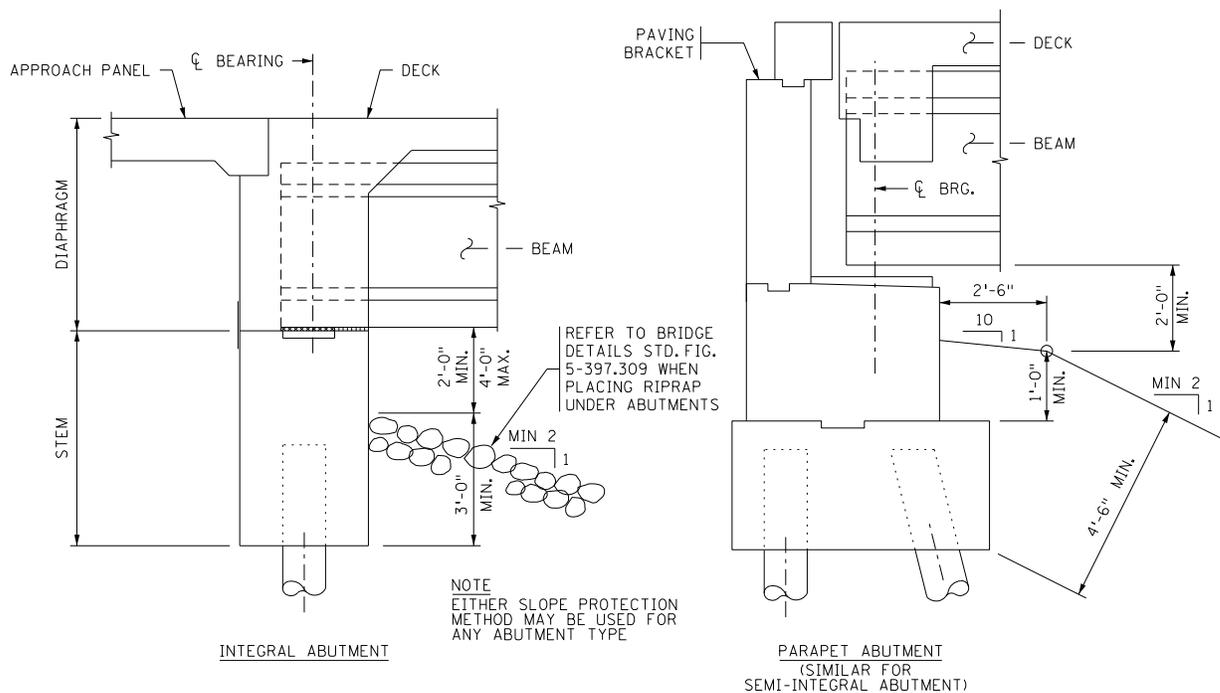
On footing details, dimension the lap splice length for bent dowel bars. For straight dowel bars, dimension the embedment or projection length.

If the railing contains a separate end post (supported on the abutment), show the end post anchorage reinforcement in the abutment details.

Membrane waterproofing (per Spec. 2481.3.B) shall be provided for construction joints, doweled cork joints, Detail B801 contraction joints, and on wall joints below ground. Waterproofing is not required at the top of parapet expansion block joints.

All reinforcement, except that completely encased in buried footings or otherwise indicated in this section, shall be epoxy coated. The minimum size for longitudinal bars in abutment and wingwall footings is #6.

Figure 11.1.2 illustrates cover and clearance requirements for abutments.



**Figure 11.1.2**  
**Cover and Clearance Requirements**

For skewed abutments, acute angles are not allowed at corners where wingwalls intersect with the abutment stem. Instead, provide a 6 inch minimum chamfer or "square up" the corner to the wingwall at all acute angle corners.

Provide shrinkage and temperature reinforcement per Article 5.2.6.

Detail sidewalk paving brackets with the same width and elevation as the roadway paving bracket. Sidewalks are to be supported on abutment diaphragm or abutment backwalls and detailed to "float" along adjacent wingwalls.

For semi-integral and parapet abutments, avoid projections on the back of abutments that are less than 4'-6" below grade. If shallower projections are necessary, slope the bottom to minimize frost heave effects.

For additional guidance on reinforcement detailing, see the web published document, *Suggested Reinforcement Detailing Practices*, which can be found at <http://www.dot.state.mn.us/bridge/standards.html>.

### **11.1.1 Integral Abutments**

An integral abutment consists of an abutment stem supported by a single line of piles. The superstructure girders or slab bear on the stem. An abutment diaphragm is poured with the deck and encases the girders. The diaphragm is connected to the stem, making the superstructure integral with the abutment. Figure 11.1.1.2 shows typical integral abutment cross-section details and reinforcement. Figure 11.1.1.3 shows typical partial elevation details and reinforcement. Figure 11.1.1.4 shows Section A-A through the partial elevation. The reinforcement in these figures is typical for an integral abutment design based on the **Integral Abutment Reinforcement Design Guide** found in this section. For abutments that do not meet the design guide criteria, these figures may not accurately reflect the final abutment design.

#### **Geometry**

Use a minimum thickness of 3 feet for the abutment stem. For skewed bridges, increase the abutment thickness to maintain a minimum of 5 inches between the beam end and the approach slab seat (See Figure 11.1.1.2). Set the abutment stem height to be as short as practical while meeting the embedment and exposure limits shown in Figure 11.1.2. The preferred abutment stem height on the low side of the bridge is 5 feet, with 3 feet below grade and 2 feet exposure. (Note that the 4'-6"

minimum depth below grade requirement for abutment footings does not apply to integral abutment stems.)

Orient H-piling such that weak axis bending occurs under longitudinal bridge movements. Limit the use of CIP piling to bridges 150 feet or less in length. Minimum pile penetration into abutment stem is 2'-6". Avoid using 16" CIP and HP 14 piles or larger because of limited flexibility.

When the angle between the back face of wingwall and back face of abutment is less than 135 degrees, provide a 2'-0" x 2'-0" corner fillet on the back face of the wingwall/abutment connection. Include the fillet along the height of the abutment stem only, stopping it at the top of the stem.

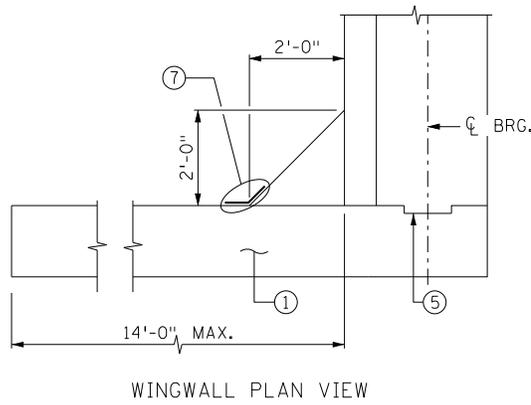
Wingwalls and the end diaphragm are intended to move as a single unit. Do not include a gap between wingwalls and the abutment diaphragm. Detail rebar to cross the joint between the diaphragm and the wingwalls.

Detail integral abutments with a drainage system (Detail B910). Outlet the 4 inch drains through wingwalls and backslopes.

Limit the length of the wingwall cantilever to 14 feet measured from the back face of abutment to the end of the wingwall.

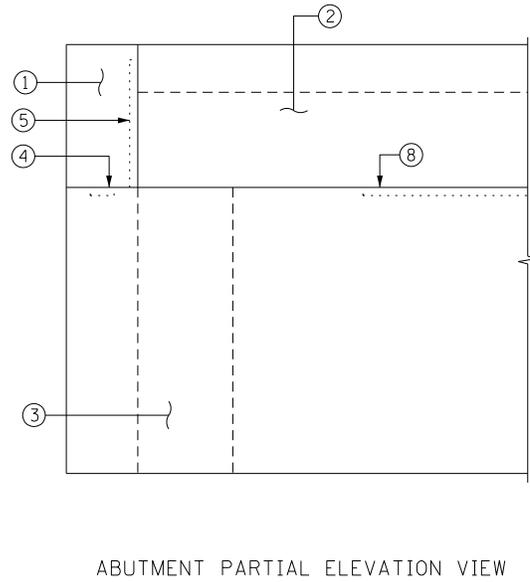
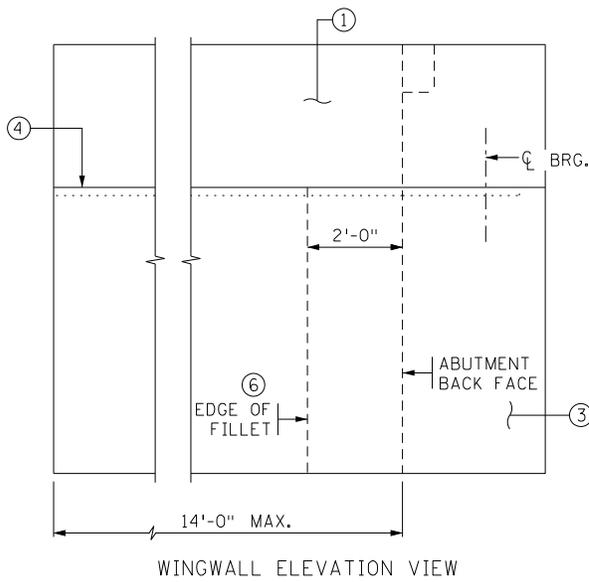
Refer to Figure 11.1.1.1a and 11.1.1.1b regarding the following guidance on integral abutment permissible construction joints. Unless indicated otherwise on the preliminary plan, place a permissible horizontal construction joint in the wingwall at the elevation of the abutment stem/diaphragm interface, running the entire length of the wingwall. For abutments with wingwalls parallel to the roadway, include a permissible vertical construction joint that is an extension of the wingwall back face through the abutment diaphragm, running from the bridge seat to the top of the wingwall. For abutments with flared wingwalls, include a permissible vertical construction joint where the wingwall connects to the abutment fillet (if provided) or abutment stem, running from the bridge seat to the top of the wingwall. Show membrane waterproofing along the inside face of all construction joints. Inclusion of these permissible construction joints allows the contractor the option of casting the upper portion of the wingwall separately or with the diaphragm and deck. Note that the upper portion of the wingwall is always to be paid for as abutment concrete, even when it is placed with the diaphragm. These permissible construction joint options may be limited for aesthetic reasons by the Preliminary Bridge Plans Engineer based on guidance from

the Bridge Architectural Specialist. In those cases, acceptable construction joint locations are to be shown on the preliminary plan.



NOTES:

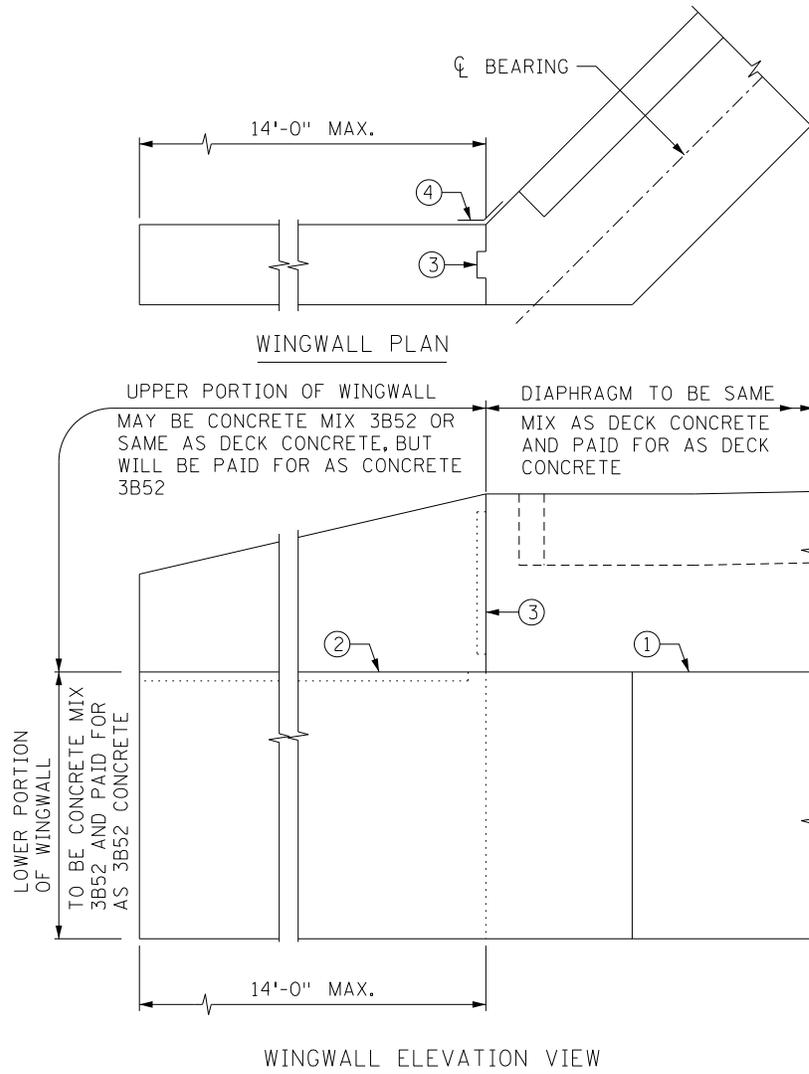
- ① UPPER PORTION OF WINGWALL MAY BE CONCRETE MIX 3B52 OR SAME AS DECK CONCRETE, BUT WILL BE PAID FOR AS 3B52 CONCRETE.
- ② DIAPHRAGM TO BE SAME MIX AS DECK CONCRETE AND PAID FOR AS DECK CONCRETE.
- ③ ABUTMENT STEM AND LOWER PORTION OF WINGWALL TO BE CONCRETE MIX 3B52 AND PAID FOR AS 3B52 CONCRETE.
- ④ PERMISSIBLE CONSTRUCTION JOINT WITH KEYWAY, IF UPPER PORTION OF WINGWALL IS PLACED WITH DIAPHRAGM AND DECK.
- ⑤ PERMISSIBLE CONSTRUCTION JOINT WITH KEYWAY, IF UPPER PORTION OF WINGWALL IS PLACED WITH ABUTMENT.
- ⑥ 2'-0" x 2'-0" FILLET EXTENDS TO TOP OF STEM.
- ⑦ MEMBRANE WATERPROOFING SYSTEM IF CONSTRUCTION JOINT IS USED.
- ⑧ CONSTRUCTION JOINT WITH KEYWAYS BETWEEN BEAMS.



**Figure 11.1.1.1a**  
**Permissible Construction Joints For Integral Abutments With**  
**Wingwalls Parallel to Roadway**

NOTES:

- ① CONSTRUCTION JOINT AT TOP OF ABUTMENT STEM WITH KEYWAYS BETWEEN BEAMS.
- ② PERMISSIBLE CONSTRUCTION JOINT WITH KEYWAY, IF UPPER PORTION OF WINGWALL IS PLACED WITH DIAPHRAGM AND DECK.
- ③ PERMISSIBLE CONSTRUCTION JOINT WITH KEYWAY (ABOVE ABUTMENT STEM), IF UPPER PORTION OF WINGWALL IS PLACED WITH ABUTMENT.
- ④ MEMBRANE WATERPROOFING SYSTEM IF CONSTRUCTION JOINT IS USED.



**Figure 11.1.1.1b**  
**Permissible Construction Joints For Integral Abutments With Flared Wingwalls**

For new bridges, tie the approach panel to the bridge with stainless steel dowel bars that extend at a 45 degree angle out of the diaphragm through the paving bracket seat and bend horizontally 6 inches below the top of the approach panel. (See bar S605S, Figure 11.1.1.2.) For repair projects, provide an epoxy coated dowel rather than stainless steel due to the shorter remaining life of the bridge. Include a ½ x 7 inch bituminous felt strip on the bottom of the paving bracket to allow rotation of the approach panel.

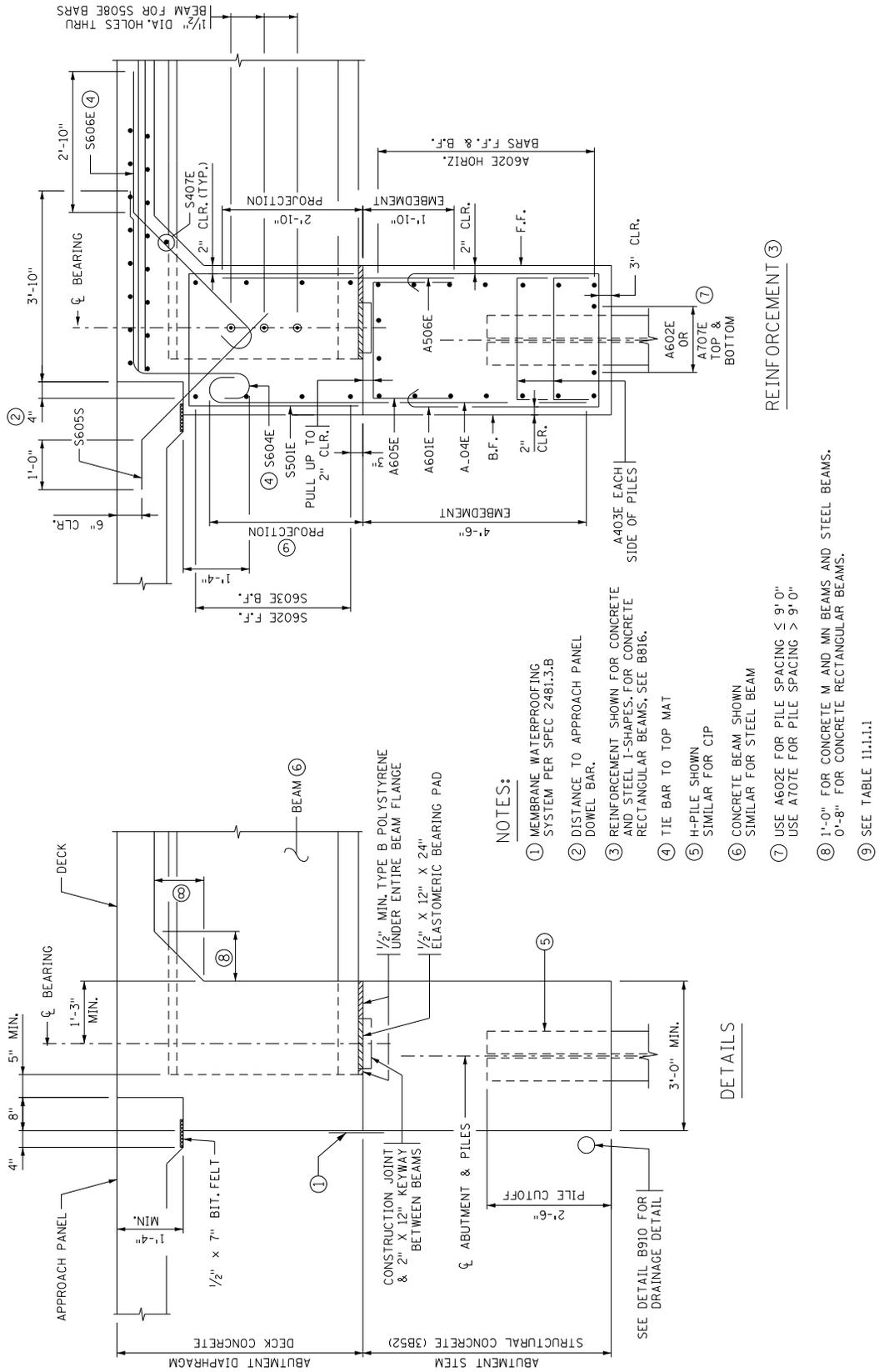


Figure 11.1.1.2

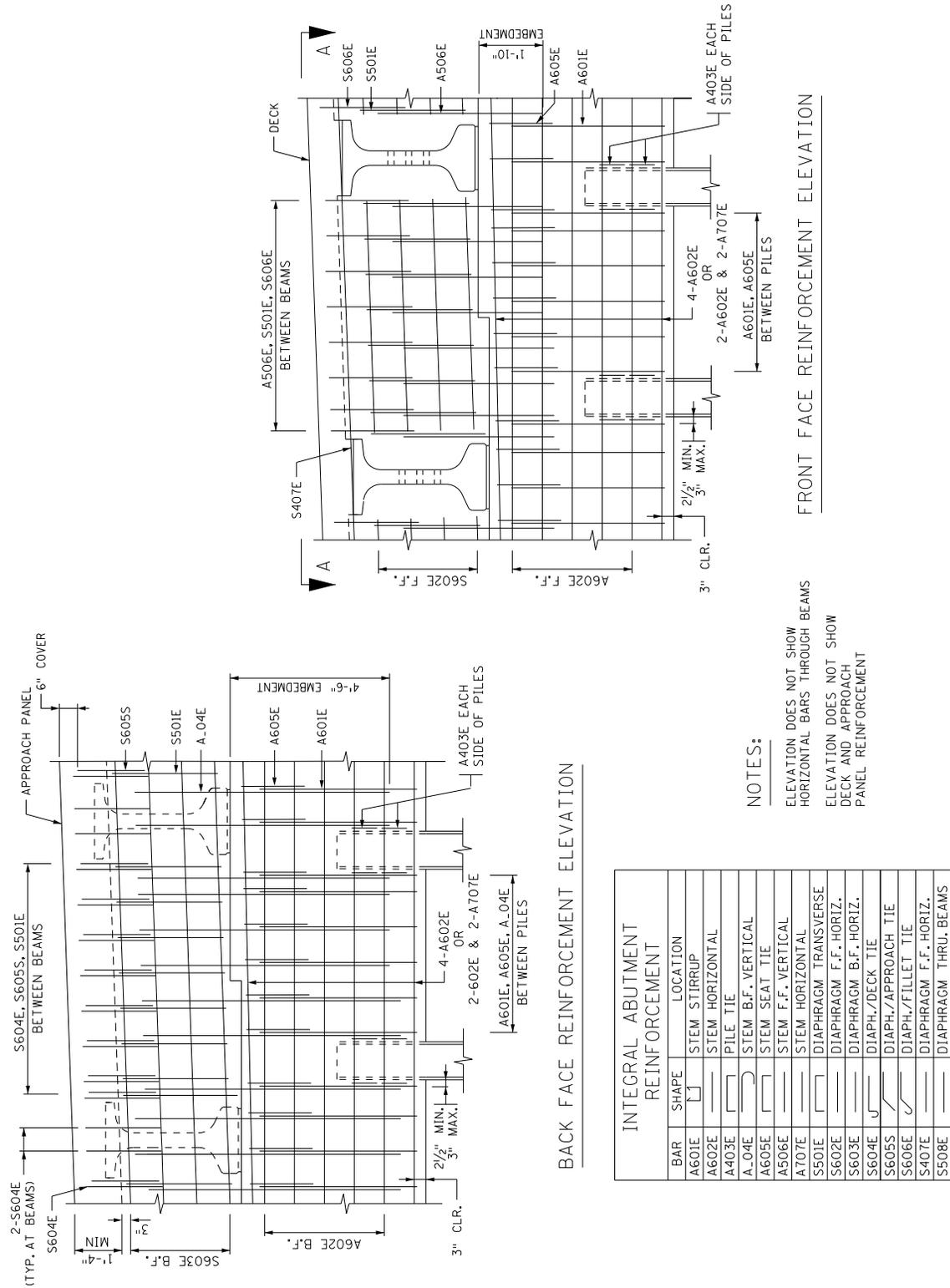
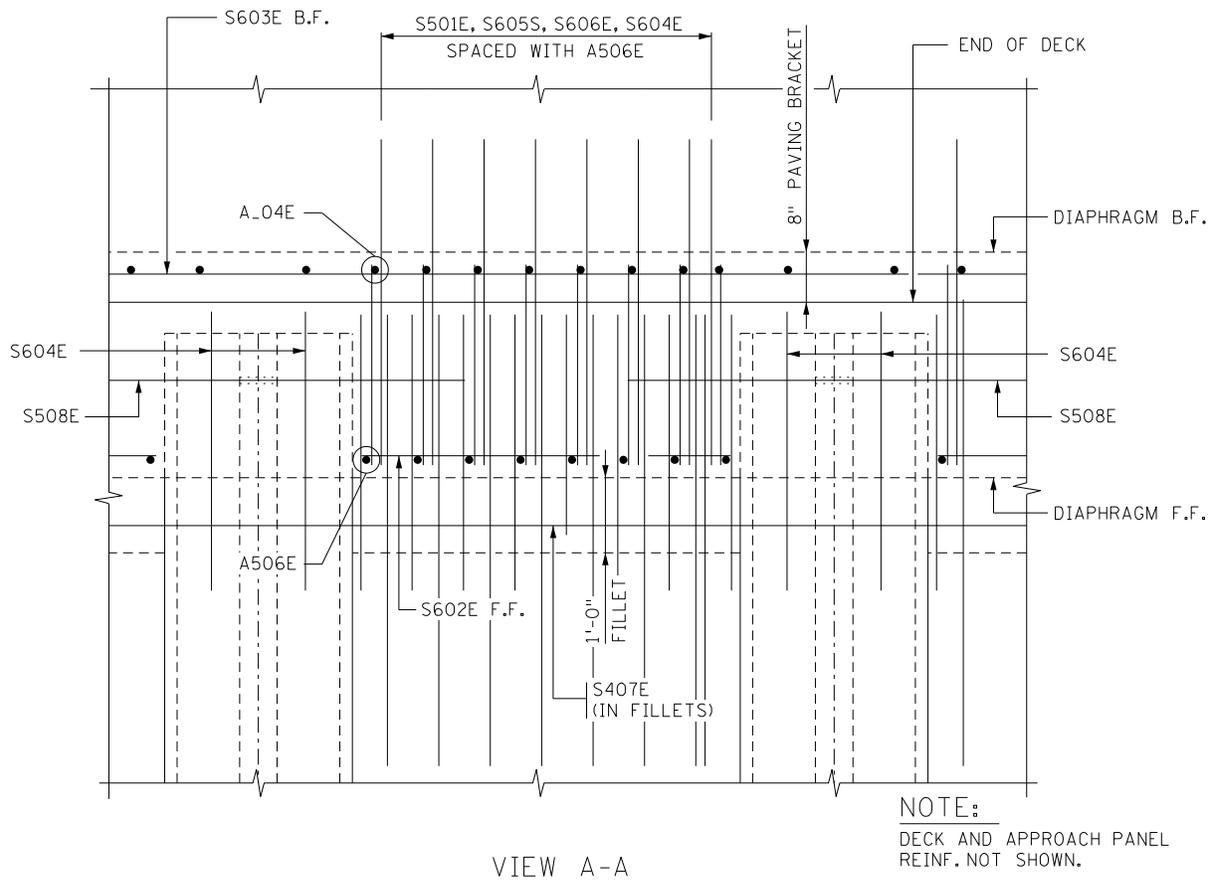


Figure 11.1.1.3

TYPICAL INTEGRAL ABUTMENT PARTIAL ELEVATION



**Figure 11.1.1.4**

**Integral Abutment Reinforcement Design Guide**

Integral abutment reinforcement may be designed using the following guidance on beam and slab span bridges where all of the following criteria are met:

- All requirements of Articles 11.1 and 11.1.1 of this manual are met
- Beam height  $\leq 72''$
- Beam spacing  $\leq 13'-0''$
- Pile spacing  $\leq 11'-0''$
- Factored pile bearing resistance  $\phi R_n \leq 165$  tons
- Maximum abutment stem height  $\leq 7'-0''$
- Deck thickness plus stool height  $\leq 15.5''$

For beam heights that fall in between current MnDOT prestressed beam sizes (i.e. steel beams), use the values corresponding to the next largest beam height in the tables. Detail reinforcement using Figures 11.1.1.2 through 11.1.1.4.

For abutment stem shear reinforcement, use #6 bars spaced at a maximum of 12 inches between piles along the length of the abutment. These bars are designated A601E and A605E in Figures 11.1.1.2 and 11.1.1.3.

For abutment stem back face vertical dowels, select bar size, spacing and length from Table 11.1.1.1. Embed dowels 4'-6" into the stem. These bars are designated A\_04E in Figures 11.1.1.2 and 11.1.1.3. Where table shows a maximum spacing of 12", space A\_04E dowels with the abutment stem shear reinforcement (A601E) between piles. Where table shows a maximum spacing of 6", space every other A\_04E dowel with the abutment stem shear reinforcement (A601E) between piles.

**Table 11.1.1.1 Abutment Stem Vertical Dowels (A\_04E) Minimum Required Bar Size and Length**

Beam Size (in)	Bar Size & Max Spacing	Bar Projection into Abutment Diaphragm
14	#5 @ 12"	8"
18	#6 @ 12"	1'-0"
22	#6 @ 12"	1'-4"
27	#6 @ 12"	1'-9"
36	#7 @ 12"	2'-6"
45	#7 @ 12"	3'-3"
54	#6 @ 6"	4'-0"
63	#6 @ 6"	4'-9"
72	#6 @ 6"	5'-6"

For abutment stem front face vertical dowels, use #5 bars spaced at a maximum of 12 inches between beams. These bars are designated A506E in Figures 11.1.1.2 through 11.1.1.4. Do not space with the other abutment stem reinforcement, but instead space with the abutment diaphragm transverse bars (S501E).

For abutment stem front and back face horizontal reinforcement, use #6 bars spaced at a maximum of 9 inches. These bars are designated A602E in Figures 11.1.1.2 and 11.1.1.3. Account for changes in abutment seat height by varying bar spacing or the number of bars.

For the abutment stem top and bottom longitudinal bars, use 4-#6 bars on the top and bottom faces of the stem for piles spaced at 9 feet or less. These bars are designated A602E in Figures 11.1.1.2 and 11.1.1.3. When pile spacing exceeds 9 feet, use #6 bars in the corners with two additional

#7 bars on the top and bottom faces of the stem. These bars are designated A602E and A707E in Figures 11.1.1.2 and 11.1.1.3.

Include 2-#4 pile ties on each side of each pile. These bars are designated A403E in Figures 11.1.1.2 and 11.1.1.3.

For abutment diaphragm transverse reinforcement, use #5 bars, which are designated S501E in Figures 11.1.1.2 through 11.1.1.4. Space them at a maximum of 12 inches between beams, matching the abutment stem front face vertical dowels (A506E).

For abutment diaphragm deck ties, approach panel ties and fillet ties, use #6 bars spaced at a maximum of 12 inches between beams to match the abutment stem front face vertical dowels. These bars are designated S604E, S605S and S606E, respectively in Figures 11.1.1.2 through 11.1.1.4. Additionally, place S604E and S605S bars outside the fascia beams to the end of the diaphragm. Do not place S606E fillet ties outside of the fascia beams. Place two additional S604E diaphragm deck ties at equal spaces at the end of each beam.

Provide 1-#4 horizontal bar in the fillet area of the abutment diaphragm that runs the width of the fillet. This bar is designated S407E in Figures 11.1.1.2 through 11.1.1.4.

For abutment diaphragm front face and back face horizontal reinforcement, use equally spaced #6 bars. These bars are designated S602E and S603E, respectively in Figures 11.1.1.2 through 11.1.1.4. Determine the number of bars using Table 11.1.1.2.

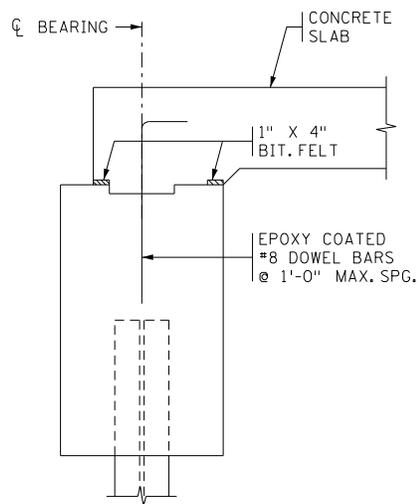
**Table 11.1.1.2**  
**Abutment Diaphragm Horizontal Bars (S602E & S603E)**  
**Minimum Required Number of #6 Bars, Each Face**

Beam Size (in)	Beam Spacing (feet)				
	≤ 9	10	11	12	13
<b>14</b>	2	2	2	2	2
<b>18</b>	2	2	2	2	2
<b>22</b>	2	2	2	2	2
<b>27</b>	3	3	3	3	3
<b>36</b>	3	3	3	3	4
<b>45</b>	4	4	4	4	5
<b>54</b>	5	5	5	5	6
<b>63</b>	6	6	6	7	7
<b>72</b>	7	7	7	8	9

For abutment diaphragms of concrete slab bridges, provide a minimum of two #6 bars in both the front face (S602E) and back face (S603E) with a maximum spacing of 12 inches.

For skews less than or equal to 20 degrees, place end diaphragm transverse bars (S501E), slab dowels (S606E), and approach panel dowels (S605S) perpendicular to the centerline of bearing. When skews exceed 20 degrees, place bars parallel to the working line.

For bridges on the local system, pinned connections between the abutment stem and diaphragm are allowed in instances where the material encountered in the soil borings for the bridge is very stable and abutment movement from slope instabilities is very unlikely. Pinned connections should be limited to concrete slab bridges with skews less than 30 degrees that have abutment stem exposure heights set at no greater than 2 feet at the low point. Provide #8 dowels at 1'-0" maximum spacing along the centerline of bearing, and a strip of 1" x 4" bituminous felt along the front edge of abutment stem and back edge of slab to allow rotation. See Figure 11.1.1.5. For all other cases, use a fixed connection similar to that shown in Figures 11.1.1.2 through 11.1.1.4.



ABUTMENT CROSS SECTION WITH PINNED CONNECTION

(STEM & SLAB REINF. NOT SHOWN)

**Figure 11.1.1.5**

**Integral Abutment General Design/Analysis Method**

Design piling for axial loads only. Assume that one half of the approach panel load is carried by the abutment. Distribute live load over the entire length of abutment. Apply the number of lanes that will fit on the superstructure adjusted by the multiple presence factor. Use a minimum of four piles in an integral abutment.

For integral abutments that do not meet the **Integral Abutment Reinforcement Design Guide** criteria found in this section, use the methods outlined below to design the reinforcement.

Design vertical shear reinforcement in the abutment stem for the maximum factored shear due to the simple span girder reactions, including the dynamic load allowance of 33%. Consider the stem to act as a continuous beam with piles as supports.

Punching shear of the piles can be assumed to be satisfied and need not be checked.

Design abutment stem backface vertical dowels for the passive soil pressure that develops when the bridge expands. Assume the abutment stem acts as a cantilever fixed at the bottom of the diaphragm and free at the bottom of the stem. Referring to Figure 11.1.1.6, determine the passive pressure  $p_p$  at the elevation of the bottom of the diaphragm and apply as a uniform pressure on the stem.

$$p_p = k_p \cdot \gamma_{\text{soil}} \cdot h_{\text{soil}}$$

$$k_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$

Where:

$k_p$  = coefficient of passive pressure

$\gamma_{\text{soil}}$  = unit weight of backfill soil

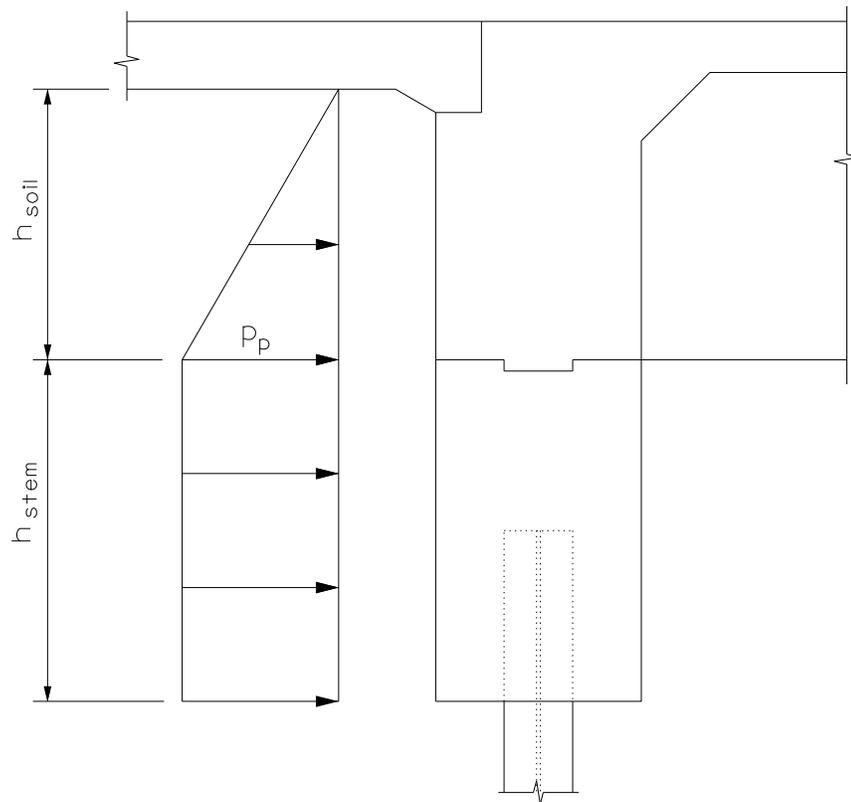
$h_{\text{soil}}$  = height of soil from top of deck to top of stem  
(see Figure 11.1.1.6)

$\phi$  = angle of internal friction of the backfill material (use 30 degrees)

Then design for a moment  $M_{up}$  equal to:

$$M_{up} = \gamma_{EH} \cdot \left( \frac{p_p \cdot h_{stem}^2}{2} \right)$$

A load factor for passive earth pressure is not specified in the LRFD specifications. Use the maximum load factor for active earth pressure,  $\gamma_{EH} = 1.50$ .



**Figure 11.1.1.6**

Design abutment stem front and back face horizontal bars for the passive soil pressure which results when the bridge expands. Consider the stem to be a continuous beam with piles as supports and design for a moment of:

$$M_{up} = \gamma_{EH} \cdot \left( \frac{w L^2}{10} \right)$$

Where:

$$\begin{aligned}w_p &= \text{passive pressure calculated at the elevation of the bottom} \\ &\quad \text{of abutment diaphragm and applied as a uniform pressure} \\ &\quad \text{on the abutment stem} \\ &= p_p \cdot h_{\text{stem}} \\ L &= \text{pile spacing}\end{aligned}$$

Design abutment stem top and bottom horizontal bars for vertical loads due to girder reactions, including dynamic load allowance of 33%. Consider the stem to be a continuous beam with piles as supports. Also, check that the front and back face horizontal bars meet the longitudinal skin reinforcement provisions of LRFD Article 5.7.3.4.

Similar to abutment stem, design abutment diaphragm horizontal bars for the passive soil pressure which results when the bridge expands. For this case, consider the diaphragm to be a continuous beam with the superstructure girders as supports.

For crack control checks, assume a Class 1 exposure condition ( $\gamma_e=1.00$ ).

For size and spacing of all other abutment diaphragm bars, refer to the **Integral Abutment Reinforcement Design Guide**.

### **11.1.2** ***Semi-Integral*** ***Abutments***

Semi-integral abutments are similar to integral abutments in that the superstructure and approach panel are connected and move together. Unlike integral abutments, the superstructure is supported on bearings that allow movement independent from the abutment stem. The abutment stem is stationary and is supported by a spread footing or a pile cap on multiple rows of piles. Figure 11.1.2.1 illustrates typical semi-integral abutment cross-section details and reinforcement.

#### **Geometry**

Skews on semi-integral abutments are limited to 30 degrees when wingwalls are parallel to the roadway in order to prevent binding of the approach panel/wingwall interface during thermal movement. For other wingwall configurations, bridge length and skew limits are the same as those for integral abutments. Whenever the skew is greater than 30 degrees, provide a concrete guide lug to limit unwanted lateral movement.

Refer to Figure 11.1.2 for minimum cover and clearance requirements. Provide a minimum abutment stem thickness of 4'-0".

Provide pedestals under the bearings and slope the bridge seat between pedestals to provide drainage toward the abutment front face. A standard seat slope provides one inch of fall from the back of the seat to the front of the seat. In no case should the slope be less than 2 percent. Set pedestals back 2 inches from front face of abutment. Minimum pedestal height is to be 3 inches at front of pedestal. Preferred maximum pedestal height is 9 inches. Provide #5 reinforcing tie bars at 6 inch to 8 inch centers in both directions under each bearing. For bearing pedestals over 9 inches tall, provide column ties in addition to other reinforcement. Provide 2 inches of clear cover for horizontal pedestal bars in the bridge seat. Provide a minimum of 2 inches of clear distance between anchor rods and reinforcing tie bars.

Provide a 3 inch minimum horizontal gap between the abutment diaphragm lug and abutment stem.

When the angle between the back face of wingwall and back face of abutment is less than 135 degrees, provide a 2'-0" x 2'-0" corner fillet on the back face of the wingwall/abutment connection. Extend the fillet from the top of footing to the top of abutment stem on the back face.

Provide a vertical construction joint at the abutment to wingwall connection. Detail the joint location with the goal of making it inconspicuous by considering the wingwall layout, abutment skew angle, fascia beam offset distance from the abutment edge, and aesthetic treatment. For wingwall layout parallel to the roadway, the preferred construction joint location is through the thickness of the abutment in a plane coincident with the back face of the wingwall. For abutments with geometry or aesthetic features that preclude this, another location such as at a vertical rustication line in the abutment or wingwall front face is appropriate. When aesthetic features govern the joint location, the Preliminary Bridge Plans Engineer will provide acceptable construction joint locations in the preliminary plan based on guidance from the Bridge Architectural Specialist. Avoid horizontal construction joints in the wingwall unless absolutely needed. If horizontal joints are needed, locate the joints at a rustication line.

Provide 1 inch of Type B (low density) polystyrene in the vertical gap between the end diaphragm and back face of wingwall. Also, provide 1 inch of Type A (high density) polystyrene in the horizontal gap between the end diaphragm lug and abutment stem. Additionally, provide a membrane waterproofing system with a 1 inch backer rod to allow movement to occur without tearing the waterproofing. Note that the membrane waterproofing and backer rod are incidental to the "Structural

Concrete (\_\_\_)" and the geotextile filter is incidental to the "Bridge Slab Concrete (\_\_\_)". See Figures 11.1.2.1 and 11.1.2.2 for details.

Place 1½ inches of Type B (low density) polystyrene between the edge of the approach panel and the back face of the wingwall to minimize binding of the approach panel on the wingwall interface during thermal movement. See approach panel standard plan sheets 5-297.225 and .229 for more details.

Detail semi-integral abutments with a drainage system behind the wall (Detail B910). Outlet the 4 inch drains through the wingwalls and backslopes.

For new bridges, tie the approach panel to the bridge with stainless steel dowel bars that extend at a 45 degree angle out of the diaphragm through the paving bracket seat and bend horizontally 6 inches below the top of the approach panel. (See bar #6S, Figure 11.1.2.1.) For repair projects, provide an epoxy coated dowel rather than stainless steel due to the shorter remaining life of the bridge. Include a ½ inch x 7 inch bituminous felt strip on the bottom of the paving bracket to allow rotation of the approach panel.

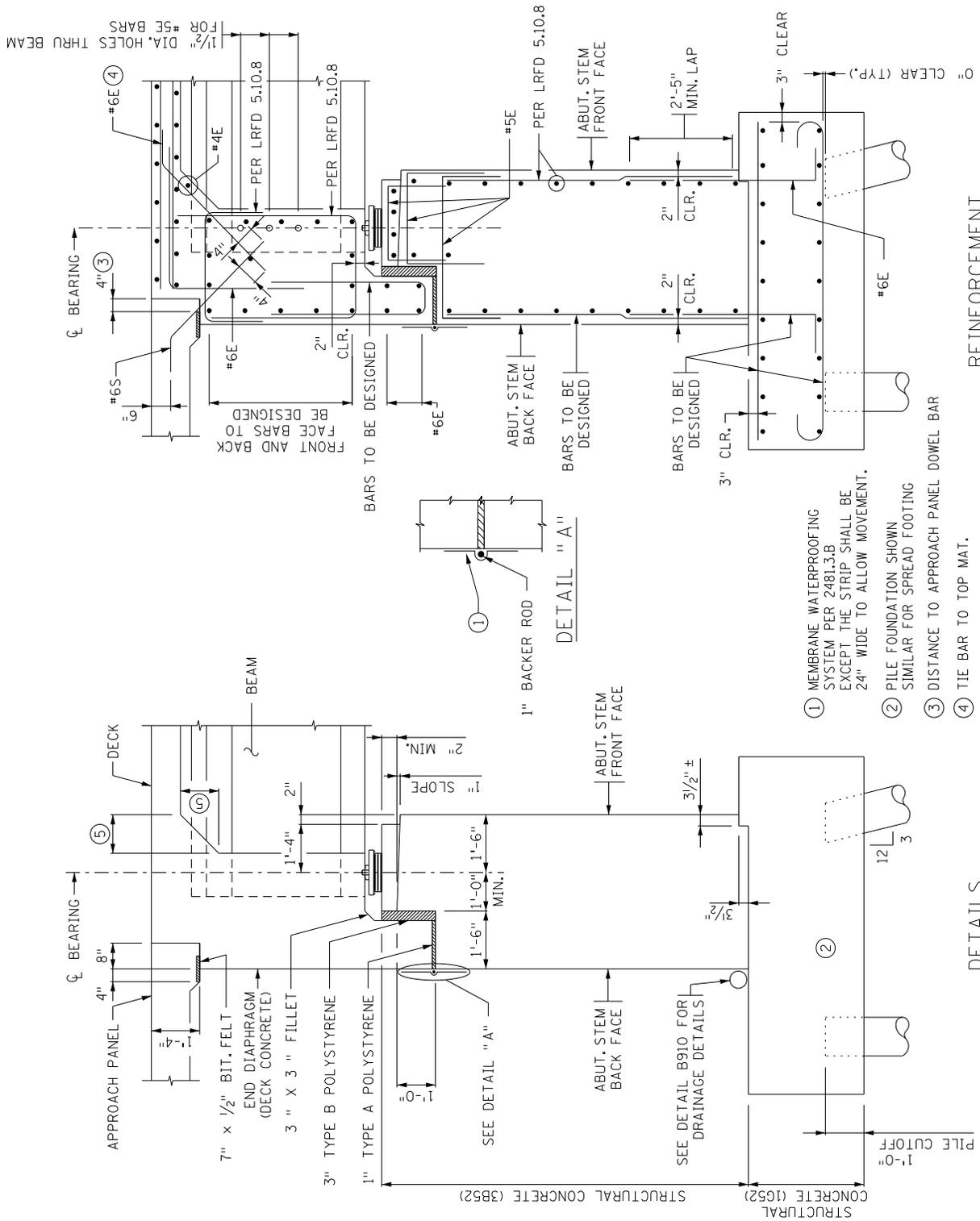
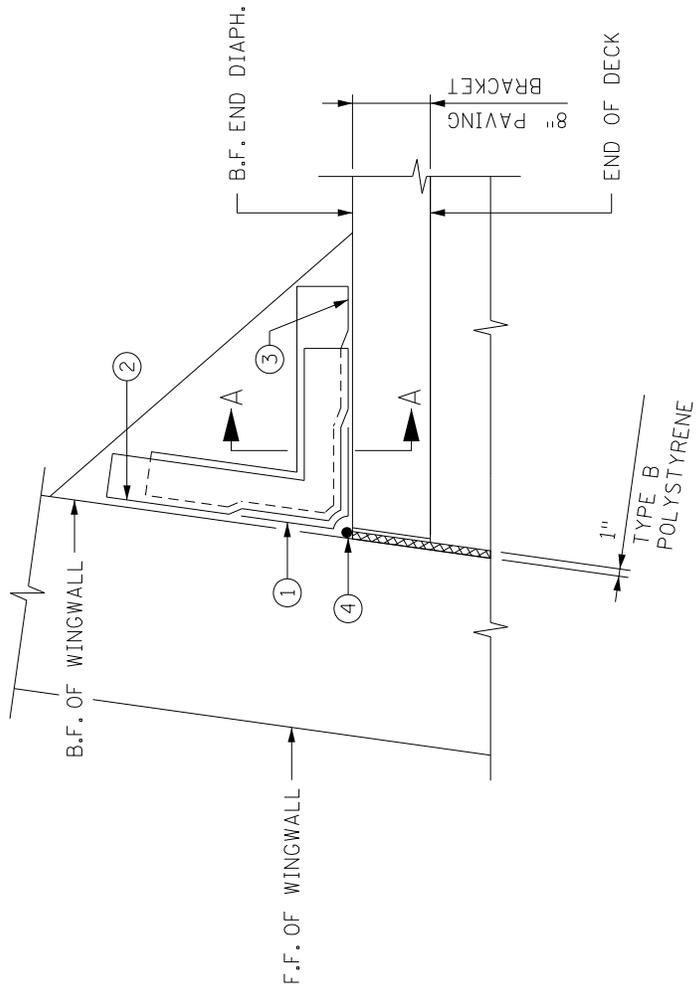
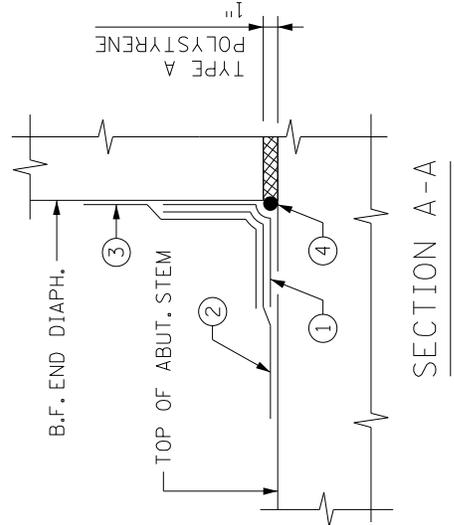


Figure 11.1.2.1



ABUTMENT PARTIAL PLAN OF  
MEMBRANE WATERPROOFING AT CORNER



NOTES:

- ① MEMBRANE WATERPROOFING SYSTEM PER SPEC. 2481.3.B
- ② GEOTEXTILE FILTER TYPE II ATTACHED TO WINGWALL. DO NOT ATTACH TO DIAPHRAGM OR OTHER LAYER, LAP 2'-0" MIN.
- ③ GEOTEXTILE FILTER TYPE II ATTACHED TO DIAPHRAGM. DO NOT ATTACH TO WINGWALL OR OTHER LAYER TO ALLOW MOVEMENT BETWEEN DIAPHRAGM & WINGWALL.
- ④ 1" BACKER ROD.

Figure 11.1.2.2

**Design/Analysis**

For single span bridges, provide fixity at one of the abutments.

Design semi-integral abutment stem, footing, and piles in accordance with Article 11.1.3 of this manual under **Design/Analysis**, except modify the Construction Case 1 loading as follows:

Construction Case 1a – Strength I (0.90DC+1.00EV+1.50EH+1.75LS)  
Abutment stem has been constructed and backfilled, but the superstructure and approach panel are not in place. Use minimum load factors for vertical loads and maximum load factors for horizontal loads. Assume a single lane (12 foot width) of live load surcharge (LS) is acting on abutments less than 100 feet long measured along the skew. Apply two lanes of LS for abutments 100 feet or longer.

Construction Case 1b – Strength I (0.90DC+1.00EV+1.50EH+1.75LS)  
Abutment has been constructed and the superstructure is in place. All of the backfill has been placed, but the approach panel has not been constructed. Use minimum load factors for vertical loads and maximum load factors for horizontal loads. Assume a single lane (12 foot width) of live load surcharge is acting on abutments less than 100 feet long measured along the skew. Apply two lanes of LS for abutments 100 feet or longer.

Design abutment diaphragm front and back face horizontal bars for the passive soil pressure which results when the bridge expands.

Design abutment diaphragm vertical bars found in the lug to resist the passive pressure that develops when the bridge expands. Assume the diaphragm lug acts as a cantilever fixed at the bottom of the diaphragm.

Semi-integral abutment diaphragm horizontal reinforcement can be designed using the **Integral Abutment Reinforcement Design Guide** found in this section, provided all of the criteria for the design guide are met. When using this guide for semi-integral abutments, the stem height requirement may be ignored. Design front and back face horizontal bars using Table 11.1.1.2, and place 4 additional #6 bars in the diaphragm lug. (See Figure 11.1.2.1).

For skews less than or equal to 20 degrees, place diaphragm transverse bars, slab dowel, and approach panel dowel bars perpendicular to the centerline of bearing. When skews exceed 20 degrees, place bars parallel to the working line.

For semi-integral abutments with total heights (stem plus footing) of less than 15 feet, use vertical contraction joints spaced at approximately 32 feet (see Detail B801). For semi-integral abutments with total heights greater than or equal to 15 feet, use construction joints (with keyways) spaced at approximately 32 feet.

### **11.1.3 Parapet Abutments**

Parapet abutments have backwall or parapet elements that are separate from the end diaphragms in the superstructure. Low parapet abutments have total heights (from top of paving block to bottom of footing) of less than 15 feet. High parapet abutments have total heights equal to or greater than 15 feet. If the total height of the abutment is more than 40 feet, counterforts should be considered.

#### **Geometry**

Refer to Figure 11.1.2 for minimum cover and clearance requirements.

When the angle between the back face of wingwall and back face of abutment is less than 135 degrees, provide a 2'-0" x 2'-0" corner fillet on the back face of the wingwall/abutment connection. Extend the fillet from the top of footing to 1 inch below the top of abutment parapet on the back face and provide a 1 inch thick polystyrene bond breaker between the top of fillet and approach panel.

Provide a vertical construction joint at the abutment to wingwall connection. Detail the joint location with the goal of making it inconspicuous by considering the wingwall layout, abutment skew angle, fascia beam offset distance from the abutment edge, and aesthetic treatment. For abutments without maskwalls that have a wingwall layout parallel to the roadway, the preferred construction joint location is at the end of the corner fillet and running through the wingwall thickness. For bridges with mask walls, the preferred construction joint location is through the thickness of the abutment in a plane coincident with the back face of the wingwall. This helps to prevent development of mask wall horizontal cracks at the top of the bridge seat that extend horizontally into the wingwall. For abutments with geometry or aesthetic features that preclude use of the preferred location, another location such as at a vertical rustication line in the abutment or wingwall front face is appropriate. When aesthetic features govern the joint location, the Preliminary Bridge Plans Engineer will provide acceptable construction joint locations in the preliminary plan based on guidance from the Bridge Architectural Specialist. Avoid horizontal construction joints in the wingwall unless absolutely needed. If horizontal joints are needed, hide the joints by locating at a rustication line.

For skews greater than 30 degrees, provide a shear lug to reduce unwanted lateral movement during bridge expansion.

Detail parapet abutment seat and pedestals in accordance with Article 11.1.2 of this manual under **Geometry**.

### **Design/Analysis**

For design of piling or footing bearing pressures, as a minimum, consider the following load cases:

Construction Case 1 – Strength I ( $0.90DC+1.00EV+1.5EH+1.75LS$ )

Abutment has been constructed and backfilled, but the superstructure and approach panel are not in place. Use minimum load factors for vertical loads and maximum load factors for horizontal loads. Assume a single lane (12 foot width) of live load surcharge is acting on abutments less than 100 feet long measured along the skew. Apply two lanes of LS for abutments 100 feet or longer.

Construction Case 2 – Strength I ( $1.25DC$ )

Abutment has been constructed, but not backfilled. The superstructure has been erected, but approach panel is not in place. Use maximum load factor for dead load.

Final Case 1 – Strength I ( $1.25DC+1.35EV+0.90EH+1.75LL$ )

Bridge is complete and approach panel is in place. Use maximum load factors for vertical loads and minimum load factor applied to the horizontal earth pressure (EH).

Final Case 2 – Strength I ( $1.25DC+1.35EV+1.50EH+1.75LL$ )

Bridge is complete and approach panel is in place. Use maximum load factor for all loads.

Design abutments for active pressure using an equivalent fluid weight of 0.033 kcf. A higher pressure may be required based on soil conditions. Neglect passive earth pressure in front of abutments.

Use LRFD Table 3.11.6.4-1 for determination of live load surcharge equivalent soil heights. Apply live load surcharge only when there is no approach panel.

Assume that one half of the approach panel load is carried by the abutment.

Distribute superstructure loads (dead load and live load) over the entire length of abutment. For live load, apply the number of lanes that will fit on the superstructure adjusted by the multiple presence factor.

For resistance to lateral loads, see Article 10.2 of this manual to determine pile resistance in addition to load resisted by battering.

Design footing thickness such that no shear reinforcement is required. Performance of the Service I crack control check per LRFD 5.7.3.4 is not required for abutment footings.

Design abutment stem and backwall for horizontal earth pressure and live load surcharge loads.

For stem and backwall crack control check, assume a Class 1 exposure condition ( $\gamma_e = 1.00$ ).

#### **11.1.3.1 Low Abutments**

Low abutments shall have vertical contraction joints at about a 32 foot spacing. (See Detail B801.)

Detail low abutments with a drainage system (Detail B910). Outlet the 4 inch drains through the wingwalls and backslopes.

Figure 11.1.3.1.1 contains typical dimensions and reinforcing for low parapet abutments.

#### **11.1.3.2 High Abutments**

High abutments shall have vertical construction joints (with keyways) at about a 32 foot spacing.

Detail high abutments with a drainage system (Detail B910). Outlet the 4 inch drains through the wingwalls and backslopes. Granular backfills at railroad bridge abutments typically includes perforated pipe drains.

Figure 11.1.3.2.1 illustrates typical high abutment dimensions and reinforcing.

#### **11.1.3.3 Parapet Abutments Behind MSE Walls**

[Future manual content]

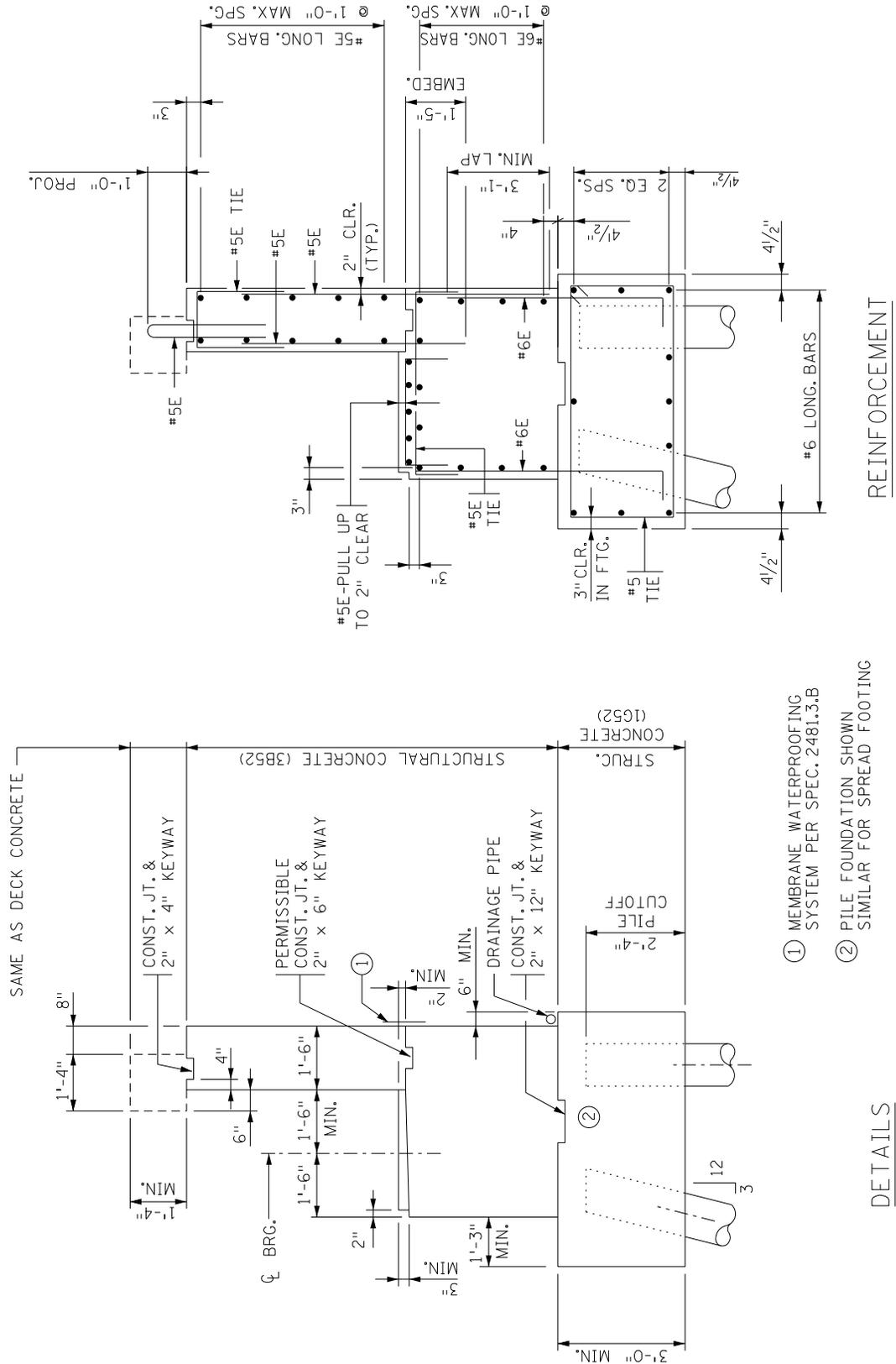
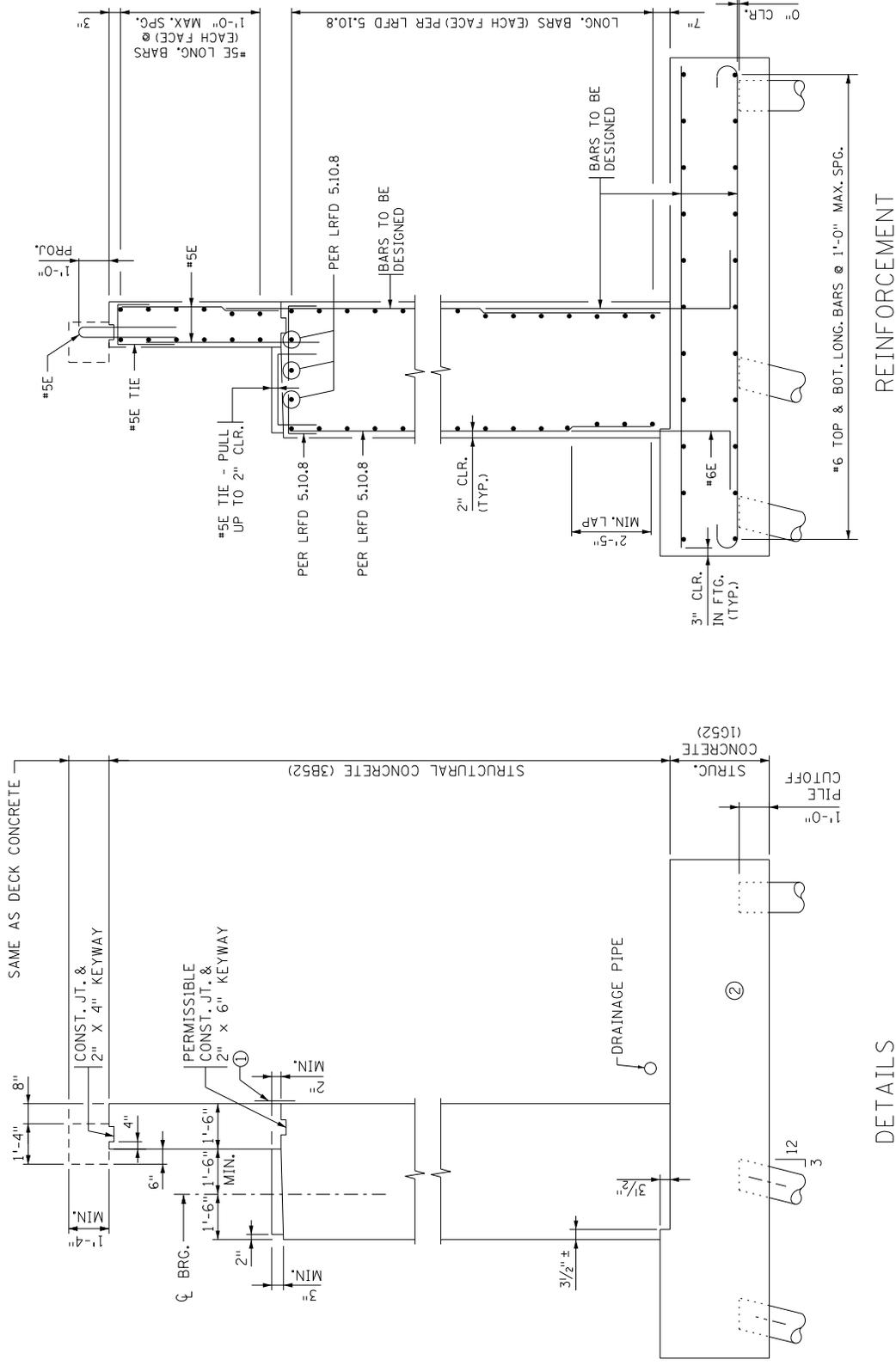


Figure 11.1.3.1.1



REINFORCEMENT

DETAILS

TYPICAL HIGH PARAPET ABUTMENT

- ① MEMBRANE WATERPROOFING SYSTEM PER SPEC. 2481.3.B
- ② PILE FOUNDATION SHOWN SIMILAR FOR SPREAD FOOTING

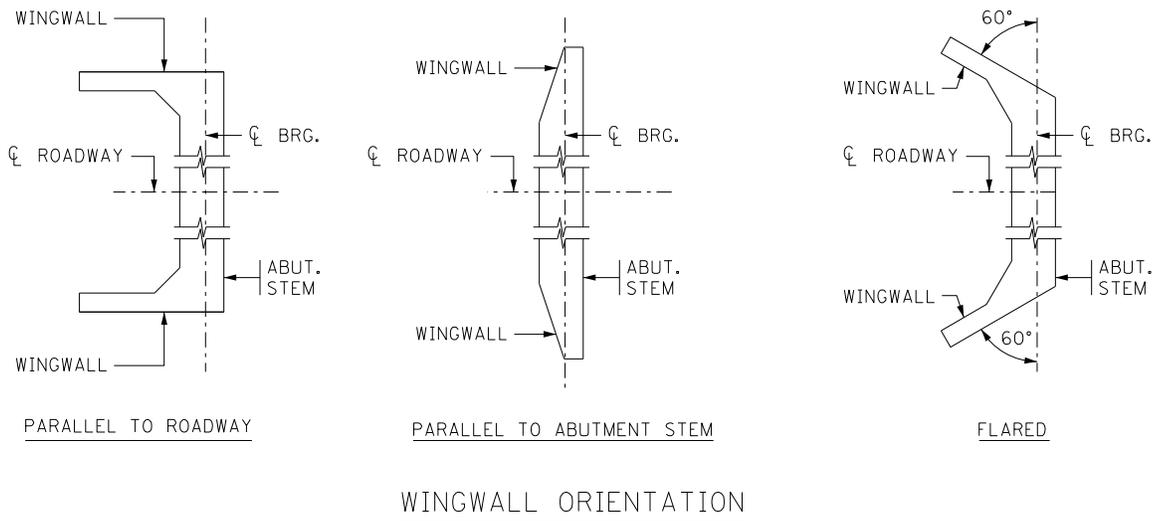
Figure 11.1.3.2.1

**11.1.4 Wingwalls**

Wingwalls are the retaining portion of the abutment structure that are located outside the abutment stem.

**11.1.4.1 Wingwall Geometry**

Wingwalls can be oriented parallel to the roadway, parallel to the abutment stem, or flared. See Figure 11.1.4.1.1. The intended orientation for the wingwalls will be provided in the Preliminary Plan. If flared, set the flare angle between the wingwall and centerline of bearing to an increment of 15 degrees.



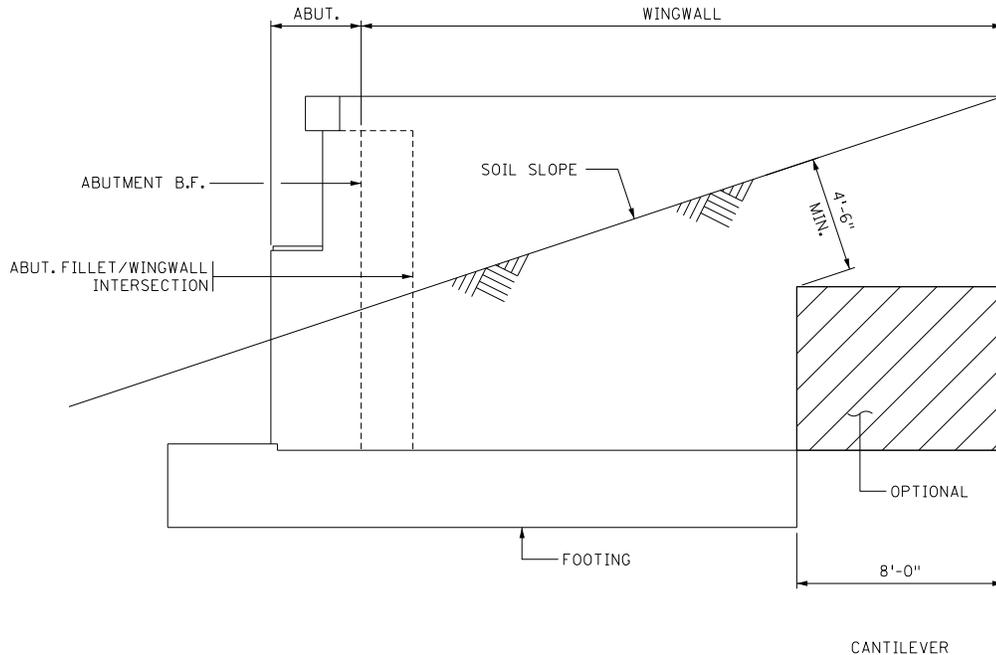
**Figure 11.1.4.1.1**

Provide a minimum wingwall thickness of 1'-6". For shorter wingwall heights, use a constant thickness. For taller wingwalls, use a wingwall thickness of 1'-6" at the top for approximately 2 feet of height to prevent binding of the approach panel if settlement occurs, and use a variable thickness in the lower portion by battering the back face at 1:24.

For integral abutments, the maximum wingwall cantilever length is 14 feet. For wingwalls oriented parallel to the roadway or flared, cantilever length is defined as the distance from the back face of abutment to the wingwall end. For wingwalls parallel to the abutment stem, cantilever length is defined as the distance from the intersection point of abutment stem and wingwall to the wingwall end. The maximum cantilever beyond the edge of footing for parapet and semi-integral abutment wingwalls is 12 feet.

The preferred wingwall layout for parapet and semi-integral abutments is shown in Figure 11.1.4.1.2. It consists of a wingwall supported by a

single footing (a continuation of the abutment footing) with an 8 foot end cantilever. The cantilever may be stepped at the end of the footing, but must be a minimum of 4'-6" below grade.



WINGWALL ELEVATION  
 PARAPET & SEMI-INTEGRAL ABUTMENT  
 PREFERRED WINGWALL LAYOUT

**Figure 11.1.4.1.2**

For parapet and semi-integral abutment wingwalls where the distance from the back face of abutment to the end of the wingwall footing is greater than 30 feet, multiple stepped or separate footings with different elevations should be considered. Generally, stepped footings are not recommended for pile foundations and separate footings are not recommended for spread footing foundations. Discuss the options with the Regional Bridge Construction Engineer.

For multiple stepped footings, use step details similar to those shown on retaining wall standard sheet 5-297.624 (2 of 3).

For multiple separate footings, use the following guidance:

- Use a maximum slope of 1 vertical on 1.5 horizontal between the bottom of footing elevations.

- Limit the cantilever (beyond the end of the footing) of wingwalls to 6 feet.
- Assume soil pressures between abutment and wingwall footing are equally distributed to both footings.

For semi-integral and parapet type abutments, avoid horizontal wingwall construction joints unless hidden by other horizontal details. Horizontal joints tend to become visible over time due to water being carried through the construction joint by capillary action. For integral abutments, see Figure 11.1.1.1 and requirements for construction joints listed in Article 11.1.1 of this manual.

Provide vertical construction joints on long wingwalls at a maximum spacing of 32 feet.

Where wingwalls are oriented parallel to the roadway, sidewalk and curb transitions should generally not be located adjacent to wingwalls.

#### **11.1.4.2 Wingwall Design**

The design process for wingwalls will depend on the abutment type and wingwall geometry. For integral abutments, the wingwall is a horizontal cantilever attached to the abutment stem with no footing support. For semi-integral and parapet abutments, the wingwalls will typically be supported by a footing for a portion of their length with a horizontal cantilever at the end.

For integral abutment wingwalls, use the following guidance:

- Design wingwalls as fixed cantilevers to resist lateral earth (EH) and live load surcharge (LS) loads.
- For wingwalls oriented parallel to the roadway, assume active soil pressure using an equivalent fluid weight of 0.033 kcf.
- For flared wingwall orientation, designing for active soil pressure may not be adequate. Depending on the bridge width, bridge length, pier fixity, and wingwall flare angle, loading from passive soil pressure should be considered.
- For wingwalls oriented parallel to the abutment stem, design for passive soil pressure loading.

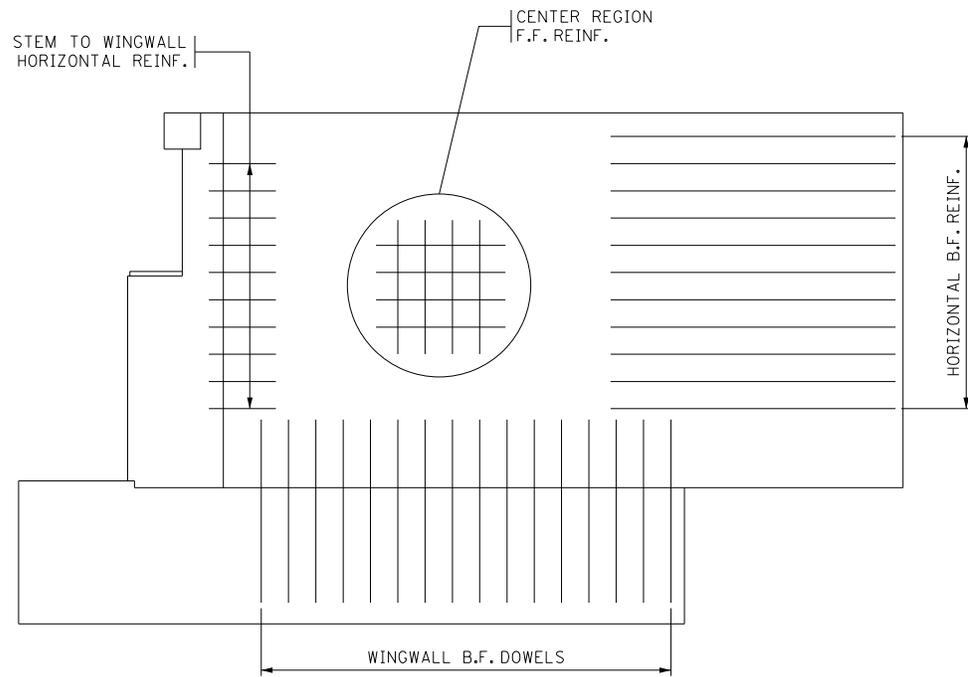
For semi-integral and parapet abutment wingwalls, use the following guidance (see Figure 11.1.4.2.1):

- Design the vertical back face wingwall dowels to resist the entire moment caused by the horizontal loads.

- Design wingwall horizontal back face reinforcement at end of footing to resist loads applied to horizontal cantilever region.
- Depending upon the wingwall height tied to the abutment stem and the length of wingwall supported by the footing, consider analyzing wingwall as a plate fixed on 2 edges to:
  - determine the stem-to-wingwall horizontal reinforcement.
  - determine the front face reinforcement in wingwall center region.

For all wingwalls with a height greater than 20 feet, a plate analysis is required.

- Provide reinforcement through the construction joint at the intersection of the wing and abutment wall to transfer wingwall loads to the abutment, if applicable.
- Within the plan set, provide wingwall pile loads if they are less than 80% of the loads in the main portion of the abutment. When listing the total length of piling for an abutment that includes a separate wingwall, check if the wingwall piles needs to be longer than the abutment piles.



WINGWALL ELEVATION  
 PARAPET & SEMI-INTEGRAL ABUTMENT  
 PREFERRED WINGWALL LAYOUT

**Figure 11.1.4.2.1**

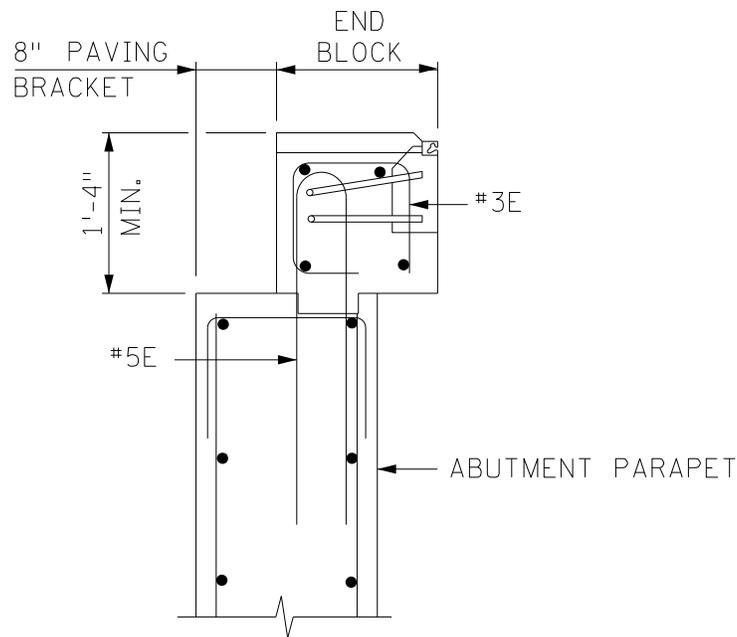
When checking crack control for wingwalls, use the Class 1 exposure condition ( $\gamma_e = 1.00$ ).

**11.1.5 Bridge Approach Panels**

Details for bridge approach panels for concrete and bituminous roadways are typically included in the roadway plans and are provided on roadway Standard Plans 5-297.222 through 5-297.231. Use a concrete wearing course on approach panels when the bridge deck has a concrete wearing course. The wearing course will be placed on the bridge superstructure and the approach panels at the same time. Therefore, include the wearing course quantity for both the approach panels and the superstructure when computing the wearing course pay item quantity for the bridge plan.

Approach panels are a roadway pay item. The preliminary bridge plan provides information to the roadway designer regarding the appropriate approach panel detail to include in the roadway plans (for a bridge with concrete barrier on the approach panel or for a bridge with concrete barrier on the wingwall). Coordinate approach panel curb and median transitions with roadway designers.

Provide 8 inches of width for the abutment paving bracket, which supports the approach panel. Place the paving bracket at 1'-4" minimum below the top of roadway surface. The reinforcement in the abutment end block is shown in Figure 11.1.5.1.



**Abutment End Block Reinforcement**  
**Figure 11.1.5.1**

**11.1.6 Bridge Approach Treatment**

For typical new bridge projects, the preliminary bridge plan provides information to the roadway designer regarding the appropriate bridge approach treatment detail to include in the roadway plans (for a bridge with integral abutments or a bridge with abutments on a footing). For repair projects and other projects where no separate grading plans are prepared, make sure that bridge approach treatments are consistent with the appropriate roadway Standard Plan 5-297.233 or 5-297.234.

**11.2 Piers**

A wide variety of pier types are used in bridge construction. The simplest may be pile bent piers where a reinforced concrete cap is placed on a single line of piling. A more typical pier type is a cap and column pier, where columns supported on individual footings support a common cap. The spacing of columns depends on the superstructure type, the superstructure beam spacing, the column size, and the aesthetic requirements. A typical cap and column pier for a roadway may have from three to five columns. At times wall piers may be used to support superstructures. Where extremely tall piers are required, hollow piers may be considered. Specialty bridges such as segmental concrete bridges may use double-legged piers to reduce load effects during segmental construction.

**11.2.1 Geometrics**

When laying out piers, consider the economy to be gained from reusing forms (both standard and non-standard) on different piers constructed as part of a single contract.

Dimension piles, footing dimensions, and center of columns to working points.

For pier caps (with cantilevers) supported on multiple columns, space the columns to balance the dead load moments in the cap.

Provide a vertical open joint in pier caps that have a total length exceeding 100 feet. The design may dictate that additional pier cap joints are necessary to relieve internal forces.

Label the ends of piers (South End, North End, etc.).

**Concrete Pier Columns**

The minimum column diameter or side of rectangular column is 2'-6".

To facilitate the use of standard forms, detail round and rectangular pier columns and pier caps with outside dimensions that are multiples of 2

inches. As a guide, consider using 2'-6" columns for beams 3'-0" or less in depth, 2'-8" columns for beams 3'-1" to 4'-0", 2'-10" columns for beams 4'-1" to 5'-0", and 3'-0" columns for beams over 5'-0" unless larger columns are necessary for strength or for adequate bearing area. Aesthetic considerations may result in larger sizes and will be provided in the Preliminary Plan.

Show an optional construction joint at the top of columns. For tall piers, consider additional intermediate permissible construction joints for constructability. All construction joints should be labeled and the size of keyways identified.

### **Concrete Pier Caps**

The preferred configuration for the top of pier caps is level or sloped with individual pedestals at each beam seat. The minimum set-back distance for pedestals is  $1\frac{1}{2}$  inches from the edge of cap. The minimum pedestal height is 3 inches. The preferred maximum pedestal height is 9 inches. When pedestal height exceeds 9 inches, consider using a stepped beam seat configuration for the pier cap.

Choose a pier cap width and length that is sufficient to support bearings and provide adequate edge distances. As a guide, choose a pier cap depth equal to 1.4 to 1.5 times the width.

The bottom of the pier cap should be approximately parallel to the top. Taper cantilever ends about  $\frac{1}{3}$  of the depth of the cap. When round pier columns are required, use rounded pier cap ends as well. The ends of pier caps for other types of pier columns should be flat. Detail solid shaft (wall) piers with rounded ends for both the cap and shaft. Aesthetic considerations may alter this guidance and will be shown in the Preliminary Plan.

Detail a  $\frac{3}{4}$  inch V-strip on the bottom of pier cap ends to prevent water from migrating on to substructure components.

### **Integral Steel Box Beam Pier Caps**

Avoid the use of steel box beam pier caps whenever possible. Conventional concrete pier caps or open plate girder pier caps are preferred.

To ensure that components are constructible, review the design details of box beam pier caps with the Fabrication Methods Unit and the Structural Metals Inspection Unit early in the plan development process.

The minimum dimensions of a box pier cap are 3'-0" wide by 4'-6" high. Make access openings within the box as large as possible and located to facilitate use by inspection personnel. The minimum size of access openings in a box pier cap is 24" x 30" (with radiused corners).

Provide access doors near each end. If possible, locate the door for ladder access off of the roadway. Orient the hinge for the access doors such that doors swing away from traffic. Access doors can be placed on the side of box pier caps if they are protected from superstructure runoff. If not, locate in the bottom of the cap. Bolt the frame for the door to the cap in accordance with Bridge Detail Part I, B942.

Bolted internal connections are preferred to welded connections. Fillet welds are preferred to full penetration welds.

Avoid details that may be difficult to fabricate due to access or clearance problems. Assume that welders need an access angle of at least 45 degrees and require 18 inches of clear working distance to weld a joint. The AISC Manual of Steel Construction contains tables with entering and tightening clearance dimensions for bolted connections.

Paint the interior of boxes for inspection visibility and for corrosion protection. Provide drainage holes with rodent screens at the low points of the box.

### **11.2.2 Pier Design and Reinforcement**

Provide 2 inches minimum clear distance between anchor rods and longitudinal reinforcement bars. For piers without anchor rods, provide a single 6 inch minimum opening between longitudinal reinforcement bars to facilitate concrete placement.

For typical pier caps, limit the size of pier cap stirrups to #5. Use open stirrups unless torsion loads are large enough to require closed stirrups. If necessary, use double stirrups to avoid stirrup spacing of less than 4 inches.

Provide #5 reinforcing tie bars at 6 inch to 8 inch centers in both directions under each bearing. For bearing pedestals over 9 inches tall, provide column ties in addition to other reinforcement. Detail ties to clear bearing anchor rods by a minimum of 2 inches.

For additional guidance on reinforcement detailing, see the web published document, *Suggested Reinforcement Detailing Practices*, which can be found at <http://www.dot.state.mn.us/bridge/standards.html>.

### 11.2.2.1 Pile Bent Piers

The preliminary plan will specify whether a pile encasement wall must be provided. An encasement wall provides stability and protects the piling from debris. Dimension encasement walls to extend from the bottom of the cap to the flowline.

For pile bent piers that do not require an encasement wall, use cast-in-place concrete (CIP) piles no smaller than 16 inches in diameter.

Design the piles to resist first and second order combined axial and bending effects under the strength limit state.

Limit deflections at the top of piles to avoid excessive movement under typical loads (not including uniform temperature effects). Choose a deflection limit that ensures the overall structure and its components will remain at a serviceable level throughout its performance life. Deflection criteria and subsequent limits shall consider number of spans, span length, span configuration, joint type, joint configuration, joint performance, bearing type, bearing layout, etc.

Consider limiting longitudinal deflections to the joint opening at the median temperature under the service limit state. Consider all loads in deflection calculations except the uniform temperature change. Deflections due to uniform temperature change are not included since they are superimposed deformations resulting from internal force effects applied to the structure and are accounted for when the joint openings are sized. Two inches is a practical limit for typical bridges.

Use the following to determine the flexural rigidity ( $EI_{\text{eff}}$ ) of CIP piles for stiffness (deflection) calculations, taken from the *AISC Steel Construction Manual*, 14<sup>th</sup> Edition, Section I2.2b.:

$$EI_{\text{eff}} = E_s I_s + E_s I_{\text{sr}} + C_3 E_c I_c$$

where  $E_s$  = elastic modulus of steel  
 $I_s$  = moment of inertia of steel pile section  
 $I_{\text{sr}}$  = moment of inertia of reinforcing bars  
 $E_c$  = elastic modulus of concrete  
 $I_c$  = moment of inertia of concrete section

$$C_3 = 0.6 + 2 \left( \frac{A_s}{A_c + A_s} \right) \leq 0.9$$

Using the above will provide flexural rigidity values between those calculated using the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* and those calculated assuming a full composite section (with concrete transformed).

Determine unbraced length by adding together the length of the pile from bottom of pier cap to ground and the assumed depth to fixity below ground.

In the direction perpendicular to the pier, use an effective length factor  $K_{\text{perp}}$  of 2.1 for analysis (fixed cantilever). In the direction parallel to the pier, use an effective length factor  $K_{\text{par}}$  of 1.2 for analysis (fixed at bottom and rotation-fixed, translation-free at the top).

Determine structural capacity for piles considering combined axial compression and flexure, and buckling. For CIP piles, determine axial resistance using AASHTO Article 6.9.5 and flexural resistance using AASHTO Article 6.12.2.3.2. Do not use the provisions of AASHTO Article 6.9.6 or 6.12.2.3.3.

Analyze the pier cap as a continuous beam supported by multiple pile supports.

For girder type superstructures, live loads are transmitted to the pier cap through the girders. Using multiple load cases, pattern the live load on the deck within the AASHTO defined lane widths to obtain maximum load effects in the pier cap. For determination of live load transmitted to the girders from the deck, assume the deck is simply supported between beam locations. Use the lever rule for exterior girders. Do not use the maximum girder reaction (computed when designing the girders) at all girder locations on the pier beam, as this will result in unrealistically high live load reactions. For piers with pile encasement walls, ignore the wall for the pier cap design.

For pier cap crack control check, assume Class 2 exposure condition ( $\gamma_e = 0.75$ ).

Use standard hooks to develop the top longitudinal reinforcement at the ends of pier caps.

For typical bridges, base the distribution of longitudinal forces to individual piers on the number of contributing fixed piers. For bridges with tall piers or long multi-span bridges, consider performing a stiffness analysis (considering pier and bearing stiffnesses) to determine the percentage of longitudinal forces distributed to each pier.

Galvanize piles from top of pile to 15 feet below ground surface to protect against corrosion.

**11.2.2.2 Cap &  
Column Type Piers**

Design pier footing thickness such that no shear reinforcement is required.

Performance of the Service I crack control check per LRFD 5.7.3.4 is not required for pier footings.

Include a standard hook at each end of all footing longitudinal and transverse reinforcement.

Use 90 degree standard hooks to anchor the dowel bars in the footing/column connection. Show the lap splice length for bent dowels and check development length of hooked end of dowel bar at footing/column interface. Unless analysis shows this is unnecessary, size dowel bars one size larger than column vertical reinforcement when the dowel bar is detailed to the inside of the column vertical.

Provide the dimensions between the center of column dowel patterns and the nearest working points.

To simplify construction, detail vertical column reinforcement to rest on top of the footing.

Use spiral reinforcement on round columns with a diameter less than or equal to 42 inches. Use a #4 spiral with a 3-inch pitch. Extend spirals no less than 2 inches into the pier cap. Use Table 5.2.2.3 to compute the weight of column spiral reinforcement.

Design round columns over 42 inches in diameter and square or rectangular columns with tied reinforcement. Use ties no smaller than #3 when the column vertical bars are #10 or smaller. Use #4 or larger ties for #11, #14, #16, and bundled column vertical bars. The maximum spacing for ties is 12 inches. Place the first tie 6 inches from the face of the footing, crash wall, or pier cap.

Design the columns to resist first and second order combined axial and bending effects under the strength limit state.

Generally, designers can conservatively use the following guidance for the distribution of longitudinal forces to individual piers:

- For fixed piers, divide the entire longitudinal force among the contributing fixed piers.
- For expansion piers, design each pier for a longitudinal force equal to the total longitudinal force divided by the total number of substructures.

Alternatively, do a stiffness analysis (considering pier and bearing stiffnesses) to determine the percentage of longitudinal forces distributed to each pier. A stiffness analysis is encouraged whenever there are 4 or more piers.

In the direction perpendicular to the pier, use an effective length factor  $K_{\text{perp}}$  of 2.1 for analysis (fixed cantilever). In the direction parallel to the pier, use an effective length factor  $K_{\text{par}}$  of 1.2 for analysis (fixed at bottom and rotation-fixed, translation-free at the top). Note that for piers with crash struts, the column length  $L$  is measured from the top of the crash strut to the bottom of the pier cap when considering loads in the direction parallel to the pier cap.

For pier caps with multiple column supports, analyze cap as a continuous beam.

For girder type superstructures, live loads are transmitted to the pier cap through the girders. Using multiple load cases, pattern the live load on the deck within the AASHTO defined lane widths to obtain maximum load effects in the pier cap. For determination of live load transmitted to the girders from the deck, assume the deck is simply supported between beam locations. Use the lever rule for exterior girders. Do not use the maximum girder reaction (computed when designing the girders) at all girder locations on the pier beam, as this will result in unrealistically high live load reactions.

For pier cap crack control check, assume Class 2 exposure condition ( $\gamma_e = 0.75$ ).

Use standard hooks to develop the top longitudinal reinforcement at the ends of pier caps.

**11.2.3 Pier  
Protection  
[3.6.5] [3.14]  
[AREMA Manual  
for Railway  
Engineering,  
Vol. 2, Ch. 8,  
Art. 2.1.5.1  
and C-2.1.5.1]**

The AASHTO LRFD Specifications includes requirements for the protection of structures against vessel and vehicle collision. The *AREMA Manual For Railway Engineering* (AREMA) includes structure protection requirements for railway train collision. The intent of the requirements is to protect bridges from collision forces that could trigger progressive collapse of the bridge.

**11.2.3.1  
Protection From  
Vessel Collision  
[3.14]**

When a bridge crosses a navigable waterway, the piers must be designed to resist a vessel collision load or be adequately protected (by fenders, dolphins, etc.) as specified in Article 3.14 of the AASHTO LRFD Specifications. See Article 3.14.2 of this manual for more information.

**11.2.3.2  
Protection  
From Vehicle &  
Train Collision  
[3.6.5]  
[AREMA Manual  
for Railway  
Engineering,  
Vol. 2, Ch. 8,  
Art. 2.1.5.1  
and C-2.1.5.1]**

When a bridge crosses a roadway or railway, the piers must be evaluated for risk of vehicle or train collision, and the design completed accordingly.

Note that due to the resistance provided by the soil behind abutment walls, abutments are considered adequate to resist collision loads and are exempt from meeting the AASHTO substructure protection requirements.

When a vehicle or train collision load occurs, lateral load will transfer to the foundation. Resistance will be provided by passive soil pressure, friction, and pile structural capacity. In addition, movement beyond what is reasonable for service loading is allowed for an extreme event situation where the survival of the bridge is the goal. Therefore, all spread footing, pile, and drilled shaft foundations are considered adequate to resist lateral collision loads and are exempt from collision load extreme event limit state analysis when the other requirements of this policy are met. Also note that when a crash strut is the proposed solution to meet the pier protection requirements, the ability of the existing foundation to carry the additional dead load of the crash strut must be considered.

Unless they meet the exemption criteria in Article 11.2.3.2.1 of this manual, pile bent piers are not allowed for use within 30 feet of roadway edges or within 25 feet of railroad track centerlines unless protected by a TL-5 barrier or approved by the State Bridge Design Engineer. In the rare case where it is allowed without barrier protection, the piles must be concrete encased and meet the "heavy construction" requirements of AREMA given in Article 11.2.3.2.2 of this manual. Design the concrete encased pile wall to resist the AASHTO 600 kip collision load. The pile foundation below ground is considered adequate as stated above and is

exempt from collision load extreme event limit state analysis. In addition, the superstructure must be made continuous over the pier to prevent loss of bearing in the event of a collision.

**11.2.3.2.1 Pier  
Protection for  
New Bridges  
Over Roadways  
[3.6.5]**

**Piers Considered Exempt From Protection Requirements**

Bridges spanning over roadways with low design speeds or minimal truck traffic are at a low risk of vehicle collision. Therefore, piers of bridges that meet either of the criteria below are not required to be protected from or designed to resist a vehicle collision:

- 1) All bridges with redundant piers where the design speed of the roadway underneath  $\leq 40$  MPH. Redundant piers are pile bent piers or piers containing continuous pier caps with a minimum of 3 columns.
- 2) All non-critical bridges with redundant piers where the design speed of the roadway underneath  $> 40$  MPH and where one of the following applies:
  - o Roadway underneath is undivided (no median) with ADTT  $< 800$
  - o Roadway underneath is divided (separated by median or barrier) and on a tangent section where it passes under the bridge and has ADTT  $< 2400$
  - o Roadway underneath is divided (separated by median or barrier) and horizontally curved where it passes under the bridge and has ADTT  $< 1200$

A critical bridge is defined as any of the following:

- o a bridge carrying mainline interstate
- o a bridge spanning over a mainline interstate
- o any bridge carrying more than 40,000 ADT (not ADTT)
- o any bridge spanning over a roadway carrying more than 40,000 ADT (not ADTT)

ADTT values stated above are based on AASHTO LRFD Table C3.6.5.1-1 and are given for two-way traffic. If ADTT values are not available, assume ADTT is equal to 10% of ADT. For both ADT and ADTT, use 20 year projected values.

All other bridge piers must be located outside the clear zone defined below, protected by a barrier, or designed to resist a vehicle collision.

### **Pier Protection Requirements for Non-Exempt New Bridges Spanning Roadways**

Bridges carrying or spanning over roadways with high design speeds and substantial traffic are at higher risk and are of major concern for vehicle collision.

All bridge piers that do not meet the criteria for "Exempt" bridges shall meet the protection requirements below for piers located within the clear zone, defined as 30 feet from the roadway edge (edge of lane) nearest the pier. Designers must also coordinate the barrier/crash strut requirements and any traffic protection requirements with the road designer. The protection options are as follows:

- Provide a crash strut designed to resist a 600 kip collision load. See Article 11.2.3.2.4 of this manual.

OR

- Design individual columns for a 600 kip collision load in accordance with AASHTO Article 3.6.5.

OR

- Protect with a 54 inch high TL-5 barrier placed within 10 feet from the face of pier or a 42 inch high TL-5 barrier placed more than 10 feet from the face of the pier. See Article 11.2.3.2.5 of this manual.

OR

- Validate that the structure will not collapse by analyzing the structure considering removal of any single column. Consider all dead load with a 1.1 load factor. Use live load only on the permanent travel lanes, not the shoulder, with a 1.0 load factor.

#### ***11.2.3.2.2 Pier Protection for New Bridges Over Railways [AREMA Manual for Railway Engineering, Vol. 2, Ch. 8, Articles 2.1.5.1 and C-2.1.5.1]***

### **Piers of New Bridges Spanning Railways**

Piers located less than 25 feet from the centerline of railroad tracks shall meet the provisions of AREMA 2.1.5.1, which requires that the piers either be of "heavy construction" or have a crash wall.

A pier is considered to be of "heavy construction" when it meets all of the following:

- The cross-sectional area of each column is a minimum of 30 square feet
- Each column has a minimum dimension of 2.5 feet
- The larger dimension of all columns is parallel to the railroad track

Crash walls must meet the following geometric requirements:

- Extend the top of the crash wall a minimum of:
  - 6 feet above top of railroad track when pier is between 12 feet and 25 feet from centerline of tracks
  - 12 feet above top of railroad track when pier is 12 feet or less from centerline of tracks
- Extend the bottom of the crash wall a minimum of 4 feet below ground line
- Extend the crash wall one foot beyond outermost columns and support on a footing
- Locate the face of the crash wall a minimum of 6 inches outside the face of pier column or wall on railroad side of pier
- Minimum width of crash wall is 2.5 feet
- Minimum length of crash wall is 12 feet

Piers of “heavy construction” and crash walls adjacent to railroad tracks shall be designed for a minimum railway collision load of 600 kips applied at an angle up to 15 degrees from the tangent to the railway. Apply the collision load at 5 feet above the top of rail elevation.

**11.2.3.2.3 Pier  
Protection for  
Existing Bridges  
Over Roadways  
[3.6.5]**

Piers of existing bridges that are part of bridge major preservation projects, bridge rehabilitation projects, or roadway repair projects may need to meet the pier protection policy requirements for new bridges given in Article 11.2.3.2.1 of this manual. The decision will be made based on the criteria found in the *Bridge Preservation Improvement Guidelines (BPIG)*.

For trunk highway bridge repair projects, the Regional Bridge Construction Engineer will coordinate with the District to determine whether a pier retrofit is required per the *BPIG*. Any requirements will then be included as part of the Bridge Repair Recommendations.

For local system bridge repair projects, the designer must coordinate with the City or County Engineer to ensure that pier retrofitting has been considered.

Note that when a crash strut is the proposed solution to meet the pier protection requirements, the ability of the existing foundation to carry the additional dead load of the crash strut must be considered.

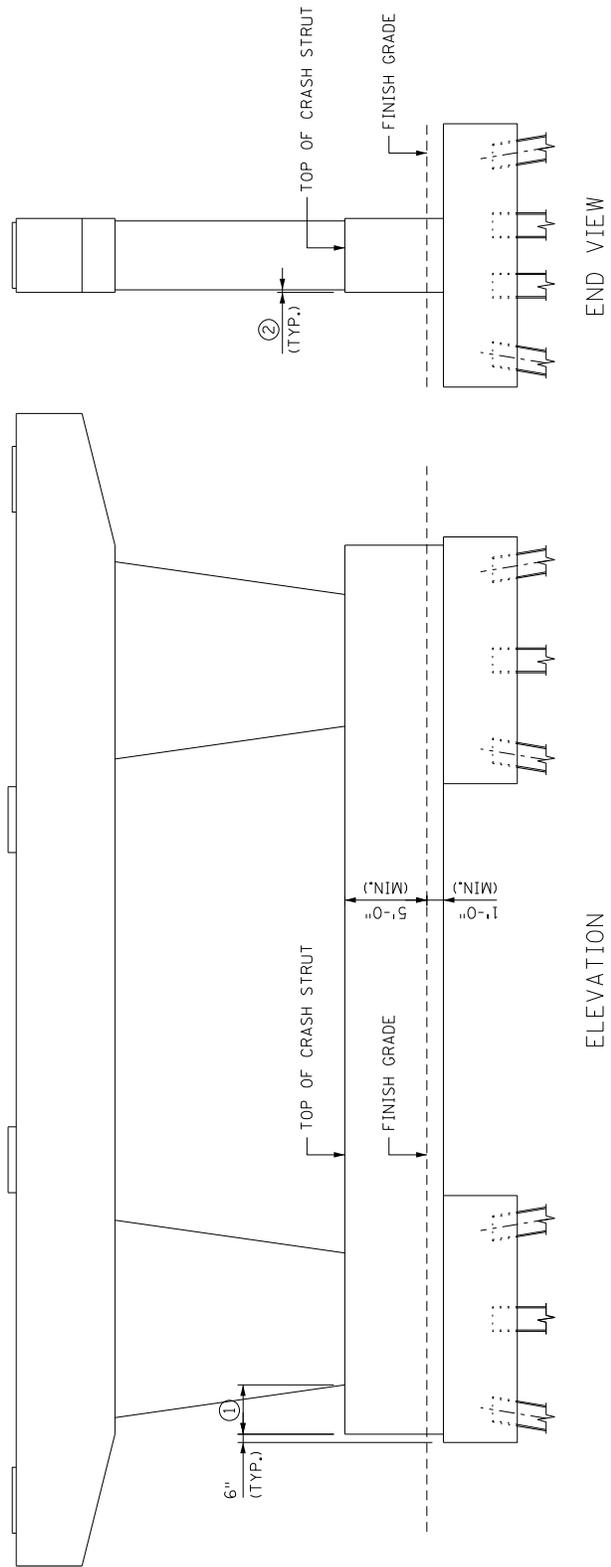
**11.2.3.2.4 Crash  
Struts for Pier  
Protection From  
Vehicle Collision****Geometry**

Refer to Figure 11.2.3.2.4.1. Extend the strut from the top of column footings to a minimum of 60 inches above the finish grade. When the strut spans between separate column footings, locate the bottom of the strut a minimum of 1 foot below the finished grade.

Provide a 3 foot minimum thickness for pier crash struts. For new pier construction, locate the strut vertical face 2 inches minimum outside of each pier column face. For pier retrofit construction, locate the strut vertical face 5 inches minimum outside of each pier column face.

A vertical face is assumed in the guidance given in this manual and is shown in all the figures. Note that an F-shape or single slope is allowed for the strut face, but will require additional strut width and detailing.

Extend the crash strut a minimum of 3 feet beyond the face of the exterior columns when a guardrail connection is required and 1 foot minimum when there is no guardrail connection. For struts that tie into a median barrier or guardrail, a vertical taper may be required at the end of the strut. Contact the MnDOT Design Standards Unit at 651-366-4622 for crash strut end taper requirements. If possible, strut to median barrier tapers should be constructed with the median barrier and located in the roadway plan. Coordinate the details with the road designer.



**NOTES:**

GUARDRAIL, END TREATMENT, OR OTHER TRAFFIC PROTECTION IS NOT SHOWN AND MUST BE COORDINATED WITH THE ROADWAY PLANS.

- ① 3'-0" MIN. WHEN GUARDRAIL CONNECTION IS REQUIRED.  
1'-0" MIN. FOR ALL OTHER SITUATIONS.  
A VERTICAL TAPER MAY BE REQUIRED AT END OF STRUT.  
CONTACT THE MDOT DESIGN STANDARDS UNIT AT 651-366-4622 FOR END TAPER REQUIREMENTS.
- ② 2" MIN. FOR NEW PIER CONSTRUCTION.  
5" MIN. FOR RETROFIT CONSTRUCTION.

**Figure 11.2.3.2.4.1  
Crash Strut Details**

### Design

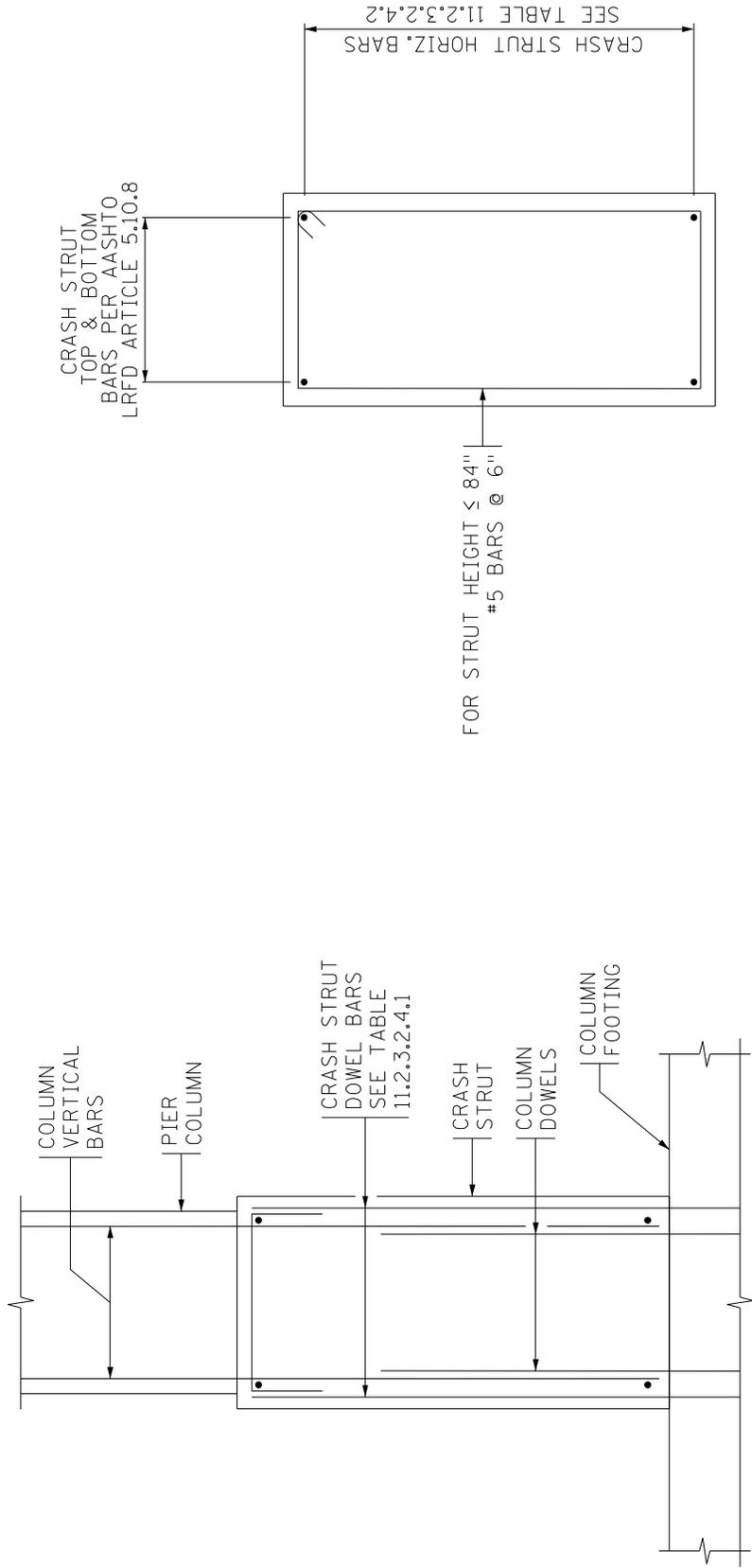
The general requirements for crash strut design are as follows:

- Design the crash strut for a 600 kip collision load applied at an angle up to 15 degrees from the tangent to the roadway.
- Apply the collision load at 5 feet above the ground line. Distribute the collision load over a length of 5 feet.
- In the column footing region, design the strut to resist the entire collision load independent of the column strength. Design the dowel reinforcement to connect the crash strut to the footing. Using yield-line theory, consider the following 2 cases:
  - Case 1) Ignoring the column strength, assume a diagonal yield-line occurs at failure. Determine crash strut capacity similar to how barrier railing capacity is determined in Section 13 of this manual.
  - Case 2) Ignoring the column strength, assume a horizontal yield-line at failure, located at the footing to crash strut interface. For this case, the strut acts as a cantilever fixed at the footing to crash strut interface and the strut capacity is based on the vertical dowels only.

Design the dowels for the case that governs. (Typically, Case 2 will govern.) Where Case 1 governs, set the length of column footing to exceed the critical yield line failure length  $L_c$  value.

- In the column footing region, assume the crash strut resists the collision load and design the column for all other loads. Extend column reinforcement through the height of the strut, detailing the collision strut reinforcement outside of the column reinforcement. Assume that the pier cap and pier strut expand and contract similarly.
- In the region between the column footings, design the strut as a simply supported horizontal beam spanning between the column footings, assuming a span length  $L$  equal to the distance between the footing edges.

Crash strut reinforcement can be determined by using the tables that follow, provided the above minimum dimensions are met. Also, refer to Figure 11.2.3.2.4.2. The tables and guidance below are for use with new construction only. On repair projects requiring a crash strut, a custom design must be completed.



BETWEEN COLUMN FOOTINGS

IN COLUMN FOOTING REGION

CROSS SECTION THROUGH CRASH STRUT

Figure 11.2.3.2.4.2

Use Table 11.2.3.2.4.1 to determine the footing to crash strut dowel reinforcement. The bar sizes and spacings were obtained by assuming that the dowel bar was fully developed at the interface of the crash strut and the top of the footing. Detail the dowel bar as necessary to ensure full development at this interface.

**Table 11.2.3.2.4.1  
Crash Strut Dowel Reinforcement for New Piers**

Strut Height Above Top of Footing (in)	Strut Thickness (in)	Length of Column Footing $L_{cs}$ Over Which the Crash Strut is Connected (ft)	
		$7 \leq L \leq 8$	$L > 8$
$\leq 84$	$\geq 36$	#7 @ 6"	#6 @ 6"

Use Table 11.2.3.2.4.2 to determine the horizontal reinforcement for the front and back face of the crash strut. Strut span length L is equal to the distance between the footing edges. Calculate the required top and bottom face horizontal bars based on the shrinkage and crack control provisions of AASHTO LRFD Article 5.10.8.

**Table 11.2.3.2.4.2  
Crash Strut Horizontal Reinforcement for New Piers**

Strut Span L (ft)	Minimum Strut Thickness (in)	$A_s$ Required on Strut Front and Back Face (in <sup>2</sup> /ft)
$\leq 10$	36	0.44
12	36	0.50
14	36	0.61
16	36	0.72
$\leq 18$	36	0.77

If the columns share a single footing and the crash strut is continuously connected to the footing, provide 0.44 in<sup>2</sup>/ft minimum horizontal reinforcement on strut front and back face.

[5.8.2.4]  
[5.8.2.5]

Shear and torsion were investigated for a 36 inch thick strut. Because shear demand exceeds 50% of  $V_c$ , and torsional forces exist, AASHTO requires minimum transverse reinforcement be provided.

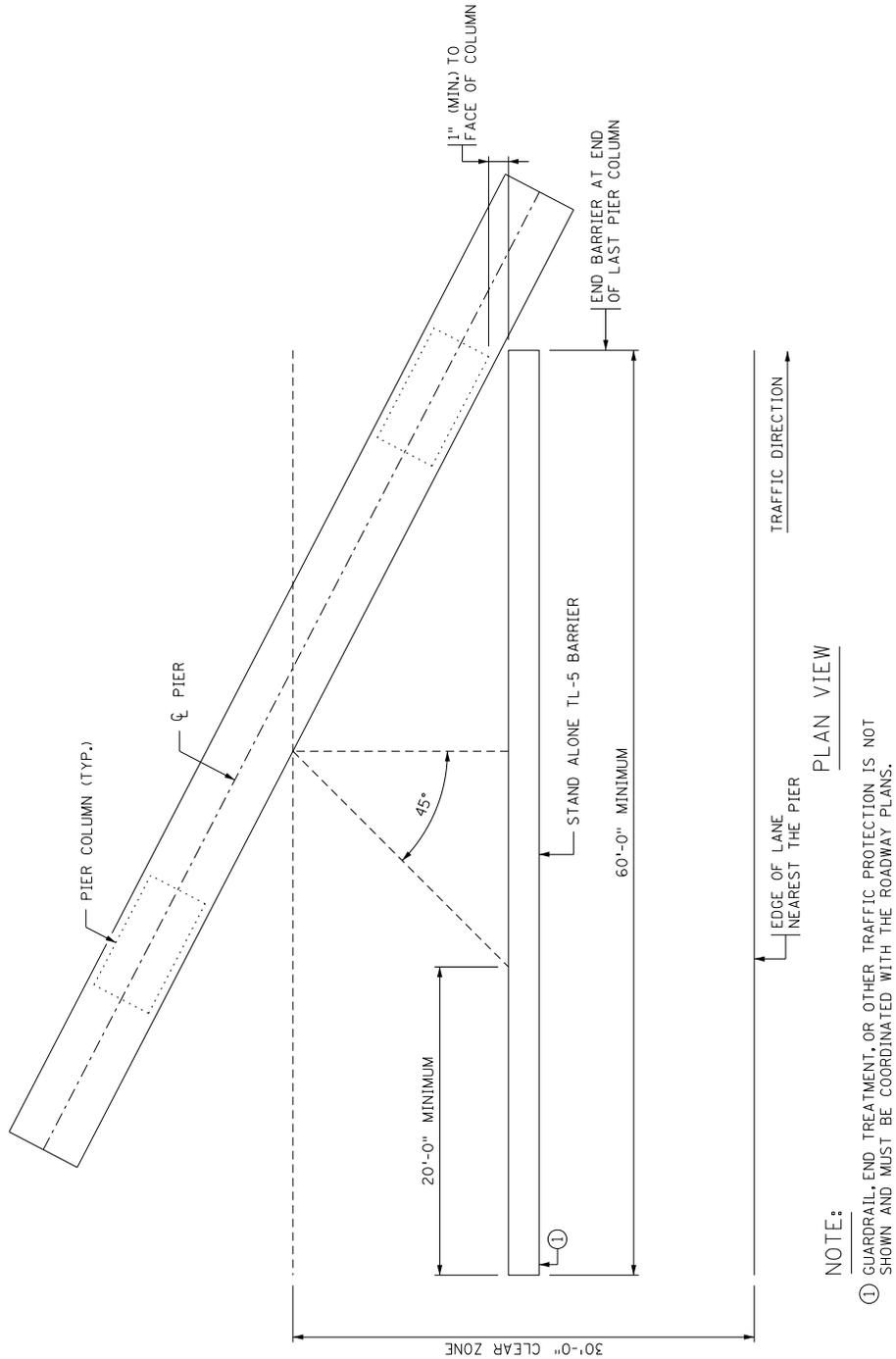
For crash strut heights up to 84 inches, provide #5 stirrup bars at 6 inch spacing. If the strut height exceeds 84 inches, calculate the minimum transverse reinforcement.

**11.2.3.2.5 Barrier  
Protection of  
Piers**

**Requirements for Test Level 5 Barrier Protection**

When the TL-5 barrier protection option is used, note that it can be tied into a concrete roadway pavement or shoulder, or it can consist of a stand-alone barrier on a moment slab. The plan layout for the barrier is dependent on the pier and roadway geometrics. (See Figure 11.2.3.2.5.1. for begin/end geometric requirements.)

Where the barrier is required to run parallel to the roadway and as close as possible to the pier, a gap is required between the back of barrier and the pier to keep the collision load from directly impacting the pier. Provide a 1 inch minimum distance between the back face of the barrier and the pier column face with polystyrene to fill the gap.



**Figure 11.2.3.2.5.1**  
**TL-5 Barrier Geometrics**

### **11.3 Retaining Walls**

The road designer will typically be responsible for leading the plan development effort for retaining walls by coordinating the wall type selection process. Several parameters must be considered for retaining wall selection and design, including:

- Height of the wall
- Geometry of the wall (curved or straight)
- Type of material retained
- Geometry of the backfill (level or sloped)
- Magnitude of live load surcharge
- Whether or not traffic barriers will be incorporated into the top of the wall (vehicle collision loads)
- Whether or not noise walls will be supported on the wall
- Location of the water table
- Quality of subgrade material (supported on spread footings or pile foundations)
- Cut or fill section
- Proximity to right of way limits

Non-standard walls, which include proprietary walls and walls not covered by available standards, require special design by the Bridge Office, a proprietary wall system engineer, or a consultant engineer. The Bridge Office has the responsibility for evaluating the structural design methodology of non-standard walls designed outside of the Bridge Office.

#### **11.3.1 Cantilever Retaining Walls**

In many cases, a conventional reinforced concrete retaining wall is the appropriate solution for a project. For wall heights up to 30 feet with level fill and up to 27 feet with live load surcharge or sloped fill with 1V:2H, use standard details. MnDOT standard cantilever retaining wall designs and details (*Roadway Standard Plans*, Fig. 5-297.620 through 5-297.639) are available for download at:

<http://standardplans.dot.state.mn.us/StdPlan.aspx>

For new wall designs that fall outside the limits of the MnDOT standards, limit the settlement of the footing to a maximum of 1 inch.

#### **[11.6.3.3]**

The current MnDOT LRFD Cast-In-Place Retaining Wall Standards were designed using the 2010 AASHTO LRFD code, for which the maximum eccentricity for foundations on soil is B/4. In the 2012 AASHTO LRFD Bridge Design Specifications, the maximum eccentricity for foundations on soil was changed to B/3. For new designs that fall outside the limits of the MnDOT standards, follow the current AASHTO requirements.

Refer to *Roadway Standard Plans*, Fig. 5-297.639 for the full basis of design for the cast-in-place retaining wall standards.

**[11.6.3.2]**  
**[10.6.5]**

For bearing checks, determine all bearing stresses using a rectangular distribution when the wall foundation is supported on soil. When the wall foundation is supported on rock, use a trapezoidal bearing stress distribution for bearing checks. For structural design of the footing, regardless of soil or rock support, always use a trapezoidal bearing stress distribution.

**11.3.2 Counterfort  
Retaining Walls**

Counterfort retaining walls may be economical for wall heights over 40 feet. Counterfort walls are designed to carry loads in two directions. Earth pressures are carried laterally with horizontal reinforcing to thickened portions of the wall. The thickened portion of the wall contains the counterfort, which is designed to contain vertical reinforcement that carries the overturning loads to the foundation.

**11.3.3 Anchored  
Walls**

**General**

Anchored walls employ earth anchors, vertical wall elements and facing. Anchored walls are used when the height of the earth to be retained by the wall is considerable and/or when all other types of retaining walls prove to be uneconomical. Anchored walls may be considered for both temporary and permanent support of stable and unstable soil and rock masses. In order to reduce the section of the stem, an anchoring system is provided at the back of the wall. Anchoring is typically accomplished by embedding a concrete block "dead man" in earth fill and connecting it to the stem of the wall with anchor rods. Alternatively, the anchors may be incorporated into soil nails or rock bolts. The feasibility of using anchored walls should be evaluated on a case-by-case basis after all other types of retaining walls have been ruled out as an option.

**Design and Construction Requirements**

Meet the current safety and movement requirements of Section 11.9 of the *AASHTO LRFD Bridge Design Specifications*.

Construction shall be in accordance with the *MnDOT Standard Specifications for Construction* and Section 7 of the *AASHTO LRFD Bridge Construction Specifications*.

**11.3.4**  
**Prefabricated**  
**Modular Block Walls**

**General**

Prefabricated modular block walls (PMBW) are gravity walls made of interlocking soil-filled concrete or steel modules or bins, rock filled gabion baskets, precast concrete units, or modular block units without soil reinforcement.

Prefabricated modular walls shall not be used under the following conditions:

- On curves with a radius of less than 800 feet, unless the curve could be substituted by a series of chords
- Steel modular systems shall not be used where the ground water or surface runoff is acid contaminated or where deicing spray is anticipated.
- Exposed heights greater than 8 feet.

**Design and Construction Requirements**

The design shall meet the current safety and movement requirements of Article 11.11 of the *AASHTO LRFD Bridge Design Specifications* and the MnDOT Division S Special Provision Boiler Plate (2411) *PREFABRICATED MODULAR BLOCK WALL (PMBW) WITH AND WITHOUT SOIL REINFORCEMENT*. The special provision can be downloaded from:

<http://www.dot.state.mn.us/pre-letting/prov/index.html>

The construction shall be in accordance with the *MnDOT Standard Specifications for Construction* and Section 7 of the *AASHTO LRFD Bridge Construction Specifications*.

**11.3.5**  
**Mechanically**  
**Stabilized Earth**  
**Walls**

**General**

Mechanically stabilized earth walls are reinforced soil retaining wall systems that consist of vertical or near vertical facing panels, metallic or polymeric tensile soil reinforcement, and granular backfill. The strength and stability of mechanically stabilized earth walls is derived from the composite response due to the frictional interaction between the reinforcement and the granular fill. Mechanically stabilized earth systems can be classified according to the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing. MnDOT uses three major types of mechanically stabilized earth walls, categorized by facing type:

1. Precast Concrete Panel (MSE) Walls: An MSE wall, in MnDOT terminology, refers to the precast concrete panel walls. Technical Memorandum No. 14-02-B-01 must be used for design and

construction of these walls. An approved list of MSE wall systems is available from the Bridge Office website.

MSE walls may be used in lieu of conventional gravity, cantilever, or counterfort retaining walls. MSE walls offer some advantages when settlement or uplift is anticipated. In some cases, MSE walls offer cost advantages at sites with poor foundation conditions. This is primarily due to the costs associated with foundation improvements such as piles and pile caps that may be required to support conventional wall systems.

In general, MSE walls **shall not** be used where:

- Two walls meet at an angle less than 70°.
- There is scour or erosion potential that may undermine the reinforced fill zone or any supporting footing.
- Walls have high curvature (radius less than 50 feet).
- Soil is contaminated by corrosive material such as acid mine drainage, other industrial pollutants, or any other condition which increases corrosion rate such as the presence of stray electrical currents.
- Sites where extensive excavation is required or sites that lack granular soils and the cost of importing suitable fill material may render the system uneconomical.
- Walls are along shorelines and are exposed to the water.
- Retaining walls support roadways unless an impervious layer is placed below the roadway surface to drain any surface water away from the reinforcement.
- There is potential for placing buried utilities within the reinforced zone.

The design of precast panel MSE walls shall meet all the requirements of the MSE Wall Technical Memorandum.

2. Modular Block Walls (MBW): The facing for this wall is made of small, rectangular dry-cast concrete units that have been specially designed and manufactured for retaining wall applications. For use of MBW, please refer to the MnDOT Technical Memorandum No. 14-03-MAT-01. MBW standard designs are shown in the Roadway Standard Plans (5-297.640, 641, 643, 644, and 645), which are available for download at:

<http://standardplans.dot.state.mn.us/StdPlan.aspx>

3. Prefabricated Wet Cast Modular Block Walls & Gabion Baskets with Earth Reinforcement: These walls are the same as described in

Article 11.3.4 except they have earth reinforcement which makes them a hybrid of a gravity wall and a MSE wall. These types of systems must be pre-qualified by the Structural Wall Committee (SWC). The maximum wall height for these walls will be set by the SWC as part of the prequalification process. The design shall meet the requirements of the MnDOT Division S Special Provision Boiler Plate (2411) *PREFABRICATED MODULAR BLOCK WALL (PMBW) WITH AND WITHOUT SOIL REINFORCEMENT*. The special provision can be downloaded from:

<http://www.dot.state.mn.us/pre-letting/prov/index.html>

Prefabricated modular walls with earth reinforcement shall not be used in the following applications:

- i. Walls supporting bridges.
- ii. Anticipated differential settlement exceeds 1/200 of the wall length.

Bidding information for prefabricated modular walls with earth reinforcement requires the preparation of plans that contain all necessary information for location and alignment including cross sections, plans, and profiles. Locations of utilities or other features impacting the design or construction must also be shown. The balance of the details necessary for construction shall be provided by the vendor via the contractor as described in the special provisions.

### **11.3.6 Noise Barriers**

Standard designs for noise barriers are covered in MnDOT Roadway Standard Plan 5-297.661. The standard plans contain detailed designs of wood planking noise barrier with concrete posts.

The panel supports used in the standard plans consist of either prestressed concrete or reinforced concrete posts.

The *MnDOT Road Design Manual* provides further information about MnDOT design and use procedures for noise barriers.

The following factors must be considered in non-standard noise barrier designs:

1. Foundation material properties such as bearing capacity, internal angle of friction, and compressibility characteristics of the surrounding soil or rock.
2. Possible ground movement.
3. Anticipated future excavation activity adjacent to the foundation.
4. Ground water level.

5. Extent of frost penetration.
6. Extent of seasonal volume changes of cohesive soils.
7. The proximity and depth of adjacent structure foundations.
8. Overall ground stability, particularly adjacent to cut or fill slopes.
9. Material properties:

Timber planking reference bending stress  $F_b = 1400$  psi

Other timber reference bending stress  $F_b = 1200$  psi

Reinforced concrete post  $f'_c = 4000$  psi

Prestressed concrete post design criteria:

Number of Strands	$f'_{ci}$ (psi)	$f'_c$ (psi)
6 or less	4000	5500
7 or more	4000	6000

10. Noise Barrier Loadings: Design of noise barrier systems shall include consideration of a variety of design loads. All possible load combinations shall be considered in the design. Such loads include:

- Dead Load - The barrier self-weight must be considered. Weight considerations are particularly critical in the design of structure-mounted barriers and may require modifications to the structure design. Lightweight barrier materials are often utilized in situations where existing or proposed structures can accommodate only a limited amount of additional weight. Ice loads represent a special type of dead load caused by water freezing and building up on exposed barrier surfaces.
- Wind Load - Wind loads vary with geographic location and can be influenced by elevation in relation to existing topography. They affect the overturning moment or rotational force placed upon the barrier, its foundation, and/or the structure to which the barrier is attached. Wind load shall meet the requirements of Section 15 of the *AASHTO LRFD Bridge Design Specifications*.
- Snow Loads - In barrier design, snow considerations relate to horizontal forces of both plowed and stored snow which can be placed against the vertical surface of the barrier. In designing the barrier to accommodate such loadings, consider the area available for safe storage of plowed snow as well as the relationship (both horizontally and vertically) of the barrier to the snow removal equipment.
- Earth Loads - In some areas, the ground elevation on both sides of the noise barrier differs and the barrier must be

designed to retain soil. Consider the possible settlement and erosion of soil on the low side of the noise wall and soil accumulation on the retained side by adding 1 foot to the design retained height of soil.

- Impact Loads - Impact loads can be classified as loads placed on the barrier due to errant vehicles and airborne debris. Apply vehicular collision forces in the design of the wall in accordance with Article 15.8.4 of the AASHTO Bridge Design Specifications. Placement of a noise barrier on a structure is usually restricted to the structure's parapet. In such cases, options for barrier mounting to the parapet (either top or face mounting) should consider the potential for impact, including the potential impact from a truck tilting into the noise barrier after hitting the protective barrier. Airborne debris loading due to retreads, stones, vehicle parts, etc., should also be considered.

11. Foundation and structural design for noise barriers shall be conducted in accordance with Section 15 of the most current *AASHTO LRFD Bridge Design Specifications*.

### **11.3.7 Cantilevered Sheet Pile Walls**

#### **General**

Cantilever sheet piling is used in many ways on bridge projects. Most often it is used to contain fill on a temporary basis for phased construction activities, as when existing embankments need protection or new embankments need to be separated from existing facilities during construction. Temporary sheet piling is also used in the construction of cofferdams. Sheet piling with concrete facing is also sometimes used in permanent wall construction.

Most often hot-rolled steel sheet piling is used for cantilevered sheet pile walls. Hot-rolled sections are available from domestic and foreign sources. Note that securing new domestic material may require a significant lead time, so check availability.

#### **Temporary Sheet Piling**

Design temporary sheet piling in accordance with the current *AASHTO Guide Specifications for Bridge Temporary Works*, and this article. Use elastic section properties for design.

For many temporary applications, new material is not required and the contractor may have a supply of used sections.

For temporary applications that are insensitive to water filtration through the interlocks, cold formed sections may be used. For railway applications, confirm with the railroad whether cold-formed sections are allowed. When cold-formed sections are used, use a reduced yield strength equal to  $0.83F_y$  to account for locked in stresses due to forming.

When an anchored wall design is required, or when significant quantities of sheet piling are anticipated (discuss with the Regional Bridge Construction Engineer to determine what is considered significant), design the wall and provide the details in the bridge plans. Include the required section modulus and tie back forces. In addition, include a lump sum pay item for the temporary sheet piling.

For most other instances, the amount and design of sheet piling used will depend on the contractor's operations. When it is anticipated that sheet pile will likely be used, show the approximate location of the sheet pile wall in the plan along with the following construction note: Payment for sheet piling shall be considered incidental to other work.

Payment for sheet piling used for typical foundation excavations is described in the standard special provisions developed for structure excavation and foundation preparation and need not be shown in the plans.

For temporary sheet piling without anchors, the deflection limit is the lesser of 1.5 inches or 1% of the exposed height. For sheet piling with anchors, the deflection limit is set to 1.0 inch. This limit may be reduced when circumstances require tighter control.

#### **Permanent Sheet Piling**

Design permanent sheet piling in accordance with the current *AASHTO LRFD Bridge Design Specifications* and this article. Use elastic section properties for design.

Do not use sheet piling for permanent wall in highly corrosive areas, defined as areas with  $\text{pH} < 5$  or  $\text{pH} > 10$ . For non-corrosive to moderately corrosive soil ( $5 \leq \text{pH} \leq 10$ ), use an effective section modulus for design determined by subtracting 0.08 inches of assumed corrosion loss (for a service life of 75 years) from the sheet pile thickness and then computing the section modulus.

For permanent sheet piling without anchors, the deflection limit is the lesser of 1.0 inch or 1% of the exposed height. For sheet pile with anchors, the deflection limit is set to 1.0 inch.

For settlement sensitive structures or where roadway pavement must be retained, the deflection limit may need to be reduced to 0.25% of the exposed height. Factors affecting the amount of reduction on the deflection limit include the following:

1. Whether existing roadway/structure integrity must be maintained.
2. Distance of wall from existing roadway or structures.
3. Type of existing roadway.
4. Height of wall or depth of excavation in front of the wall.
5. Soil type retained by the wall and to some degree the type of soil removed from in front of the wall.
6. Material and geometric properties of the wall.
7. The wall system's ability to undergo distortion & retain functionality.
8. Construction sequencing with regards to refurbishing/repaving the existing roadway relative to construction of wall.

#### **11.4 Design Examples**

Section 11 concludes with three design examples. The examples are a high parapet abutment supported on piling, a retaining wall supported on a spread footing, and a three column pier.

**11.4.1**  
**High Parapet**  
**Abutment**  
**Design Example**

This example illustrates the design of a high parapet abutment using the following procedure:

- Determine material and design parameters
- Determine loads and load combinations
- Design abutment piling
- Design abutment pile footing
- Design abutment stem and backwall
- Design wingwalls

The design parameters for the example include the following:

- 1) This example is a continuation of the prestressed I-beam and fixed bearing design examples found in Articles 5.7.2 and 14.8.1, respectively, of this manual. The superstructure consists of a 9" deck on six MN63 beams with a beam spacing of 9'-0" and no skew.
- 2) The abutment is supported on 12-inch diameter cast-in-place piling. The footing elevation was set to provide a minimum cover of 4'-6". The stem was set at the standard 4'-6" thickness to provide a 3'-0" wide seat and a 1'-6" thick backwall. Assuming a 1" minimum concrete bearing pedestal at the front of the backwall, a 3.25" bearing, a 4.75" stool height, and a 0.02 ft/ft cross slope, an average backwall height of 5'-9" was chosen for design.
- 3) The abutment supports half of a 20'-0" long approach panel which is 1'-0" thick. The approach panel supports a 20'-0" long concrete barrier on each side. Also, an abutment end block which measures 1'-4" wide by 1'-4" high is attached at the top of the backwall.

A typical cross-section for the abutment is provided in Figure 11.4.1.1. Other material and design parameters are presented in Table 11.4.1.1.



**A. Evaluate Pile Bearing Capacity**

The Bridge Construction Unit's foundation recommendations are referenced at the start of final design. The recommendations identify the pile type and factored pile bearing resistance to be used in design:

- Pile Type: 12" diameter x ¼" cast-in-place concrete
- Factored Pile Bearing Resistance,  $\phi R_n = 100$  tons/pile  
= 200 kips/pile

Figure 11.4.1.2 shows a plan view of the abutment and includes an assumed pile layout for the example. Pile rows I, II and III each contain eight piles. Generally, try to avoid pile layouts that permit individual piles to go into tension.

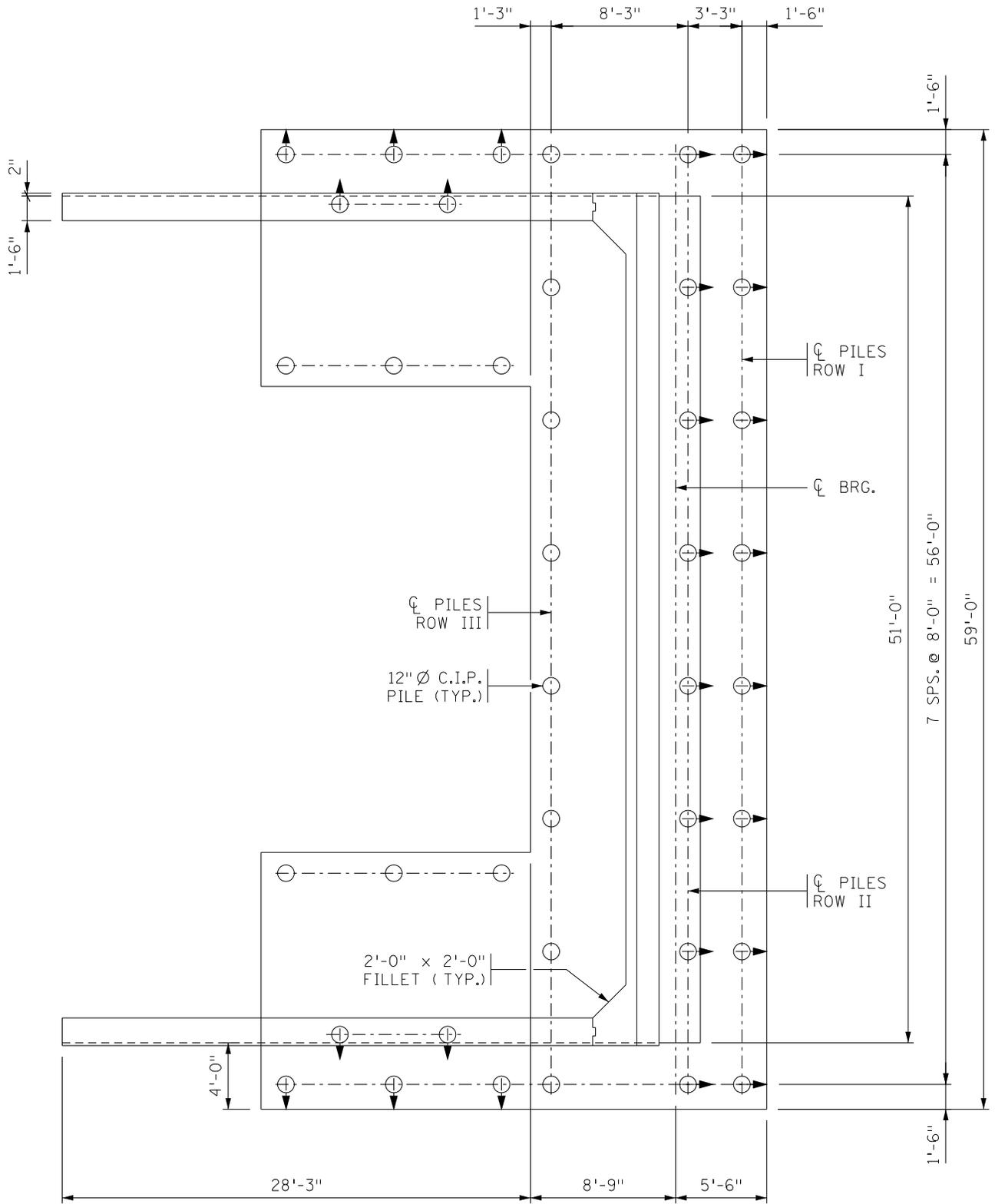


Figure 11.4.1.2

**B. Permanent Loads (DC & EV)**

Calculate the unfactored dead loads:

Superstructure Dead Load:

The vertical reaction is taken from Table 5.7.2.4 of the prestressed I-beam example:

$$P_{\text{super}} = 156 \cdot (6 \text{ girders}) = 936.0 \text{ kips}$$

Backwall:

$$P_{\text{bw}} = 0.150 \cdot 1.50 \cdot 5.75 \cdot 51 = 66.0 \text{ kips}$$

Stem:

$$P_{\text{st}} = 0.150 \cdot 4.5 \cdot 15.75 \cdot 51 = 542.2 \text{ kips}$$

Beam Seat Pedestals:

Assuming pedestals are 3.5 feet wide with an average height of 3 inches,

$$P_{\text{ped}} = 0.150 \cdot 2.83 \cdot 3.50 \cdot 0.25 \cdot 6 = 2.2 \text{ kips}$$

Footing:

To simplify load calculations, weight of the step under the stem is included with the stem.

$$P_f = 0.150 \cdot (3.5 \cdot 10.25 + 3.75 \cdot 4) \cdot 59 = 450.2 \text{ kips}$$

Approach Panel:

Assuming half the load is carried by the abutment,

$$P_{\text{ap}} = 0.150 \cdot 1 \cdot 20 / 2 \cdot 48 = 72.0 \text{ kips}$$

Abutment End Block:

$$P_{\text{eb}} = 0.150 \cdot 1.33 \cdot 1.33 \cdot 51 = 13.5 \text{ kips}$$

Wingwall DL:

Include the dead load only from that portion of the wingwall that lies on the 5'-9" heel of the abutment footing. The rest of the wingwall dead load will be incorporated into the wingwall design as it is resisted by the wingwall. The corner fillet weight is minimal and can be neglected.

$$P_{\text{wing}} = 0.150 \cdot 2 \cdot 1.50 \cdot 5.75 \cdot (15.75 + 5.75 + 1.00) = 58.2 \text{ kips}$$

Barrier DL:

The barrier on the deck is already accounted for in the superstructure dead load. Only include the additional barrier load located on the end block and approach panel or wingwalls. In this case, the barrier is located on the approach panel.

$$P_{\text{apbar}} = 0.439 \cdot 2 \cdot (0.5 \cdot 20 + 1.33) = 9.9 \text{ kips}$$

Summing the dead loads,

$$\begin{aligned} P_{\text{DC}} &= 936.0 + 66.0 + 542.2 + 2.2 + 450.2 + 72.0 + 13.5 + 58.2 + 9.9 \\ &= 2150.2 \text{ kips} \end{aligned}$$

Calculate the unfactored vertical earth pressure (EV) of fill above the footing:

On the Heel:

$$P_{\text{EV(heel)}} = 0.120 \cdot (15.75 + 5.75) \cdot 5.75 \cdot 48 = 712.1 \text{ kips}$$

On the Toe:

$$P_{\text{EV(toe)}} = 0.120 \cdot [(3.35 + 1.35) / 2] \cdot 4 \cdot 59 = 66.6 \text{ kips}$$

**C. Earth Pressure (EH)**  
[3.11.5]

The active earth pressure values used for the equivalent fluid method (described in LRFD Article 3.11.5.5) range from 0.030 kcf to 0.040 kcf. Assuming a level backfill, MnDOT practice is to use:

$$\gamma_{\text{eq}} = 0.033 \text{ kcf}$$

The respective horizontal active earth pressures at the top and bottom of the abutment are:

$$P_{\text{top}} = 0 \text{ ksf}$$

$$P_{\text{bottom}} = \gamma_{\text{eq}} \cdot h = 0.033 \cdot 25.00 = 0.825 \text{ ksf}$$

$$P_{\text{EH}} = 0.5 \cdot 0.825 \cdot 25.00 \cdot 48 = 495.0 \text{ kips}$$

The force acts at a location of  $\frac{1}{3}$  times the height of the load:

$$\text{arm} = \frac{25.00}{3} = 8.33 \text{ ft}$$

Passive earth pressure in front of the abutment is neglected in the design.

**D. Live Load Surcharge (LS)**  
[3.11.6]

The live load surcharge is applied to the abutment during construction. It represents construction activity on the fill behind the abutment prior to construction of the approach panel.

$$\Delta p = \gamma_{\text{eq}} \cdot h_{\text{eq}}$$

From Table 3.11.6.4-1, since the height of soil for vehicular loads is greater than 20 feet, use a surcharge height of 2.0 feet.

$$\Delta p = 0.033 \cdot 2.0 = 0.066 \text{ kips/ft}^2$$

MnDOT practice is to use a 12.0 foot width in determining the live load surcharge for abutments that are less than 100.0 feet in length along the skew. This is equal to the surcharge from a single lane of vehicular live load.

Horizontal Resultant of LS is:

$$P_{LS} = 0.066 \cdot 25.00 \cdot 12 = 19.8 \text{ kips}$$

The force acts at a location of  $1/2$  times the height of the load:

$$\text{arm} = \frac{25.00}{2} = 12.50 \text{ ft}$$

### **E. Live Load (LL)**

The maximum live load reaction without dynamic load allowance can be determined using Table 3.4.1.2 from this manual. For a 137 foot span:

$$R_{LL} = 66.8 + 41.6 + \frac{7}{10} \cdot (67.2 - 66.8 + 44.8 - 41.6) = 110.9 \text{ kips/lane}$$

Coincident with live load on the superstructure, lane loading is applied to the approach panel. Use the same distribution that was used for dead load (assume that one half of the total load is carried by the abutment and the other half is carried in direct bearing to the subgrade away from the abutment):

$$R_{LLapp} = 0.64 \cdot 20 \cdot \frac{1}{2} = 6.4 \text{ kips/lane}$$

### **[Table 3.6.1.1.2-1]**

For maximum loading, four lanes of traffic are placed on the superstructure and approach panel. The multiple presence factor for more than 3 design lanes is 0.65. For simplicity, add the live load from the approach panel to the live load from the superstructure and apply the total at the centerline of bearing:

$$P_{LL} = (110.9 + 6.4) \cdot 4 \cdot 0.65 = 305.0 \text{ kips}$$

Figure 11.4.1.3 summarizes the loads and includes moment arms in parentheses measured from the toe of the footing. The loads, moment arms, and moments are also tabulated in Tables 11.4.1.2 and 11.4.1.3.

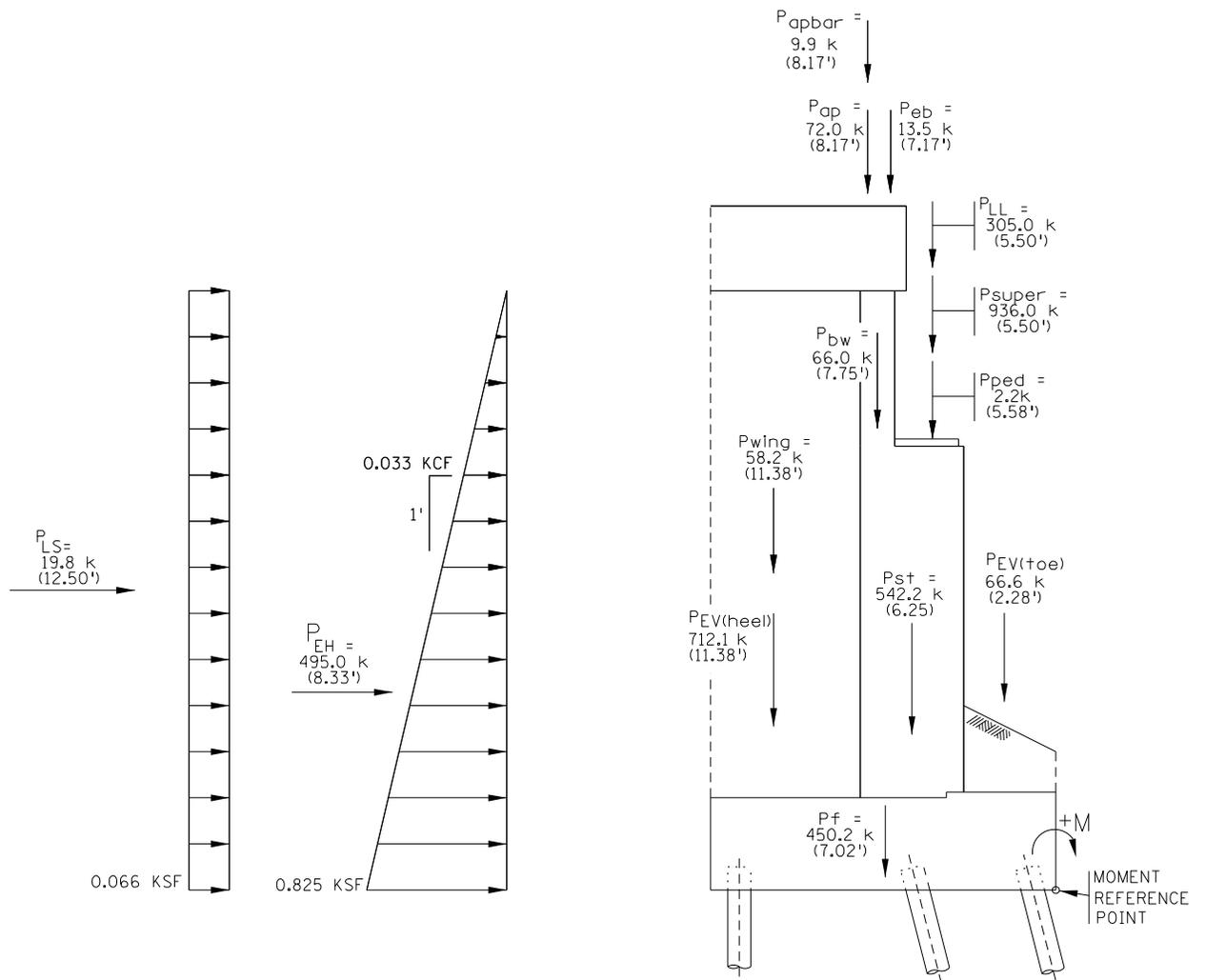


Figure 11.4.1.3

**Table 11.4.1.2 Unfactored Vertical Load Components and Moments about Toe of Footing**

	Load	Label	P (kips)	Distance To Toe (ft)	Moment About Toe (kip-ft)
DC	Superstructure DL	P <sub>super</sub>	936.0	-5.50	-5148.0
	Backwall	P <sub>bw</sub>	66.0	-7.75	-511.5
	Stem	P <sub>st</sub>	542.2	-6.25	-3388.8
	Beam Seat Pedestals	P <sub>ped</sub>	2.2	-5.58	-12.3
	Footing	P <sub>f</sub>	450.2	-7.02	-3160.4
	Approach Panel	P <sub>ap</sub>	72.0	-8.17	-588.2
	End Block	P <sub>eb</sub>	13.5	-7.17	-96.8
	Wingwall	P <sub>wing</sub>	58.2	-11.38	-662.3
	Barrier	P <sub>apbar</sub>	9.9	-8.17	-80.9
		<b>Total</b>		<b>2150.2</b>	
EV	Backfill on Heel	P <sub>EV(heel)</sub>	712.1	-11.38	-8103.7
	Fill on Toe	P <sub>EV(toe)</sub>	66.6	-2.28	-151.8
		<b>Total</b>	<b>778.7</b>		<b>-8255.5</b>
LL	Live Load	P <sub>LL</sub>	305.0	-5.50	-1677.5

**Table 11.4.1.3 Unfactored Horizontal Load Components and Moments about Bottom of Footing**

Load			H (kips)	Distance to Toe (ft)	Moment About Toe (kip-ft)
Type	Description	Label			
EH	Horizontal Earth Load	P <sub>EH</sub>	495.0	8.33	4123.4
LS	Live Load Surcharge	P <sub>LS</sub>	19.8	12.50	247.5

**F. Select  
Applicable Load  
Combinations and  
Factors For Pile  
Design  
[1.3.3 - 1.3.5]  
[3.4.1]**

The following load modifiers will be used for this example:

Load Modifier Type	Strength	Service
Ductility, $\eta_D$	1.0	1.0
Redundancy, $\eta_R$	1.0	1.0
Importance, $\eta_I$	1.0	n/a
$\eta = \eta_D \cdot \eta_R \cdot \eta_I$	1.0	1.0

Assemble the appropriate load factor values to be used for each of the load combinations. Load combinations for the Strength I limit state are used. The load cases considered for the design example are:

Strength I: Construction Case 1

$$0.90 \cdot DC + 1.00 \cdot EV + 1.50 \cdot EH + 1.75 \cdot LS$$

For this construction case, DC does not contain any dead load from the superstructure, approach panel, or abutment end block. It also assumes that the abutment is backfilled prior to superstructure erection.

Strength I: Construction Case 2

$$1.25 \cdot DC$$

For this construction case, DC includes the superstructure but does not include the approach panel. It assumes the superstructure is erected prior to the abutment being backfilled.

Strength I: Final Case 1

$$1.25 \cdot DC + 1.35 \cdot EV + 0.90 \cdot EH + 1.75 \cdot LL$$

This load case represents the completed structure with the minimum load factor for the horizontal earth pressure load.

Strength I: Final Case 2

$$1.25 \cdot DC + 1.35 \cdot EV + 1.50 \cdot EH + 1.75 \cdot LL$$

This load case represents the completed structure with the maximum load factor for the horizontal earth pressure load.

Table 11.4.1.4 contains the load factors that are used for each load component for each load case.

**Table 11.4.1.4 – Load Factors**

Load	Load Component	Load Combination			
		Strength I: Constr. 1	Strength I: Constr. 2	Strength I: Final 1	Strength I: Final 2
DC	$P_{super}$	-	1.25	1.25	1.25
	$P_{bw}$	0.90	1.25	1.25	1.25
	$P_{st}$	0.90	1.25	1.25	1.25
	$P_{ped}$	0.90	1.25	1.25	1.25
	$P_f$	0.90	1.25	1.25	1.25
	$P_{ap}$	-	-	1.25	1.25
	$P_{eb}$	-	1.25	1.25	1.25
	$P_{wing}$	0.90	1.25	1.25	1.25
	$P_{apbar}$	-	-	1.25	1.25
EV	$P_{EV(heel)}$	1.00	-	1.35	1.35
	$P_{EV(toe)}$	1.00	-	1.35	1.35
EH	$P_{EH}$	1.50	-	0.90	1.50
LS	$P_{LS}$	1.75	-	-	-
LL	$P_{LL}$	-	-	1.75	1.75

**G. Design Piles**  
**[10.7.1.5]**

Table 11.4.1.5 lists the net vertical, horizontal, and moment forces that are applied to the pile group for each of the four load combinations.

**Table 11.4.1.5 – Force Resultants**

	Vertical Load P (kips)	Horizontal Load H (kips)	Moment about Toe $M_{toe}$ (kip-ft)
Strength I: Construction Case 1	1786	777	-8599
Strength I: Construction Case 2	2585	0	-16,225
Strength I: Final Case 1	4273	446	-27,431
Strength I: Final Case 2	4273	743	-24,957

**Check Vertical Capacity of Pile Group**

Determine the properties of the pile group. These properties include the number of piles, the location of the centroid or neutral axis with respect to the toe, and the moment of inertia of each pile row.

**Table 11.4.1.6 – Pile Group Properties**

Pile Group Properties	Row Number			Sum
	I	II	III	
Piles Per Row N	8	8	8	24
Distance to Toe $d_{toe}$ (ft)	1.50	4.75	13.00	
$N \cdot d_{toe}$ (ft)	12.00	38.00	104.00	154.00
Neutral Axis of Pile Group to Toe $X_{NA}$ (ft)	$(\sum N \cdot d_{toe}) / \sum N$			6.42
Distance from Neutral Axis to Pile Row $c$ (ft)	4.92	1.67	-6.58	
$I = N \cdot c^2$ (ft <sup>2</sup> )	193.7	22.3	346.4	562.4

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.

The force in each pile row is found using:

$$\text{Pile load} = \frac{P}{N} + \frac{M_{NA} \cdot c}{I}$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group. For Strength I: Construction Case I, the eccentricity about the toe is

$$e_{toe} = M_{toe} / P = -8599 / 1786 = -4.81 \text{ ft}$$

Then the eccentricity about the neutral axis of the pile group is

$$e_{NA} = X_{NA} + e_{toe} = 6.42 - 4.81 = 1.61 \text{ ft}$$

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P \cdot e_{NA} = 1786 (1.61) = 2875 \text{ kip} - \text{ft}$$

Then Pile Load<sub>RowI</sub> =  $1786/24 + 2875 \cdot 4.92/562.4 = 99.6$  kips/pile

$$\text{Pile Load}_{\text{RowII}} = 1786/24 + 2875 \cdot 1.67/562.4 = 83.0 \text{ kips/pile}$$

$$\text{Pile Load}_{\text{RowIII}} = 1786/24 + 2875 \cdot (-6.58)/562.4 = 40.8 \text{ kips/pile}$$

The same calculations were carried out for the other load cases.

A summary of  $M_{NA}$  and the pile loads are provided in Table 11.4.1.7.

**Table 11.4.1.7 – Factored Pile Loads**

Load Combination	Eccentricity about toe $e_{toe}$ (ft)	Eccentricity about N.A. $e_{NA}$ (ft)	Moment about N.A. of pile group $M_{NA}$ (kip-ft)	Pile Loads $P_u$ (kips/pile)		
				Row I	Row II	Row III
Strength I: Construction Case 1	-4.81	1.61	2875	99.6	83.0	40.8
Strength I: Construction Case 2	-6.00	0.42	1086	117.2	110.9	95.0
Strength I: Final Case 1	-6.42	0.00	0	178.0	178.0	178.0
Strength I: Final Case 2	-5.84	0.58	2478	199.7	185.4	149.0

The largest pile load  $P_u$  occurs in Row I:

$$P_u = 199.7 \text{ kips} < 200 \text{ kips} \quad \text{OK}$$

The reduction in maximum pile bearing resistance due to the 3:12 pile batter is minimal and can be ignored.

Therefore, the pile layout is considered satisfactory for bearing.

**Check Lateral Capacity of Pile Group**

The maximum factored horizontal load from Table 11.4.1.5 is

$$H = 777 \text{ kips}$$

From Table 10.2.1 of this manual, assume a factored horizontal resistance,  $\phi R_{nh}$  of 24 kips/pile plus the resistance due to the two rows of battered piles.

$$\phi R_{nh} = 24 (24) + [8(99.6) + 8(83.0)] \left( \frac{3}{\sqrt{3^2 + 12^2}} \right) = 930 \text{ kips}$$

$$\phi R_{nh} = 930 \text{ kips} > 777 \text{ kips} \quad \text{OK}$$

**Pile Load Table for Plan**

Piles are driven until the field verification method used indicates the pile has reached refusal or the required nominal pile bearing resistance indicated in the plan. The pile bearing resistance is verified in the field

using either the MnDOT Pile Formula 2012 (MPF12) or the Pile Driving Analyzer (PDA) as described in Article 10.2 of this manual. Designers must calculate the required nominal pile bearing resistance for the controlling load and show it in the plan using the Standard Plan Note table for abutments with piling (see Appendix 2-H of this manual).

For Strength I: Final Case 2,

$$P_{LL} = 1.75 (305.0) = 533.8 \text{ kips}$$

$$M_{LL} = 1.75 (-1,677.5) = -2,935.6 \text{ kip - ft}$$

$$e_{toe,LL} = \frac{M_{LL}}{P_{LL}} = \frac{-2935.6}{533.8} = -5.50 \text{ ft}$$

$$e_{NA,LL} = x_{NA} + e_{toe,LL} = 6.42 - 5.50 = 0.92 \text{ ft}$$

$$M_{NA,LL} = P_{LL} \cdot e_{NA,LL} = 533.8 (0.92) = 491.1 \text{ kip - ft}$$

$$\begin{aligned} \text{Pile Load}_{Row I,LL} &= 533.8/24 + 491.1 (4.92)/562.4 = 26.5 \text{ kips / pile} \\ &= 13.3 \text{ tons/pile} \end{aligned}$$

$$\begin{aligned} \text{Pile Load}_{Row I,DL} &= 199.7 - 26.5 = 173.2 \text{ kips / pile} \\ &= 86.6 \text{ tons/pile} \end{aligned}$$

The final results to be shown in the plan are:

<b>ABUTMENT</b>	
<b>COMPUTED PILE LOAD - TONS/PILE</b>	
FACTORED DEAD LOAD + EARTH PRESSURE	86.6
FACTORED LIVE LOAD	13.3
* FACTORED DESIGN LOAD	99.9

\* BASED ON STRENGTH I LOAD COMBINATION.

ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES $R_n$ – TONS/PILE		
FIELD CONTROL METHOD	$\Phi_{dyn}$	* $R_n$
MNDOT PILE FORMULA 2012 (MPF12)  $R_n = 20\sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	199.8
PDA	0.65	153.7

\*  $R_n = (\text{FACTORED DESIGN LOAD}) / \Phi_{dyn}$

**H. Check Shear in Footing**

General practice is to size the thickness of footings such that shear steel is not required. Try a 42 inch thick footing with a 3 inch step at the toe.

**Determine  $d_v$**

Based on past design experience assume the bottom mat of steel is #8 bars spaced at 12 inches ( $A_s = 0.79 \text{ in}^2/\text{ft}$ ). The effective shear depth of the section ( $d_v$ ) is computed based on the location of the flexural reinforcement. The piling has an embedment depth of one foot. MnDOT practice is to place the bottom mat of reinforcement directly on top of piling embedded one foot or less. Therefore, of the three criteria for determining  $d_v$ , MnDOT does not use the  $0.72h$  criterion in this case because the flexural reinforcement location is so high above the bottom of the footing.

[5.8.2.9]

Begin by determining the depth of flexural reinforcement:

$$d_{toe} = (\text{footing thickness}) - (\text{pile embedment}) - (d_{bar} / 2)$$

$$= 45 - 12 - 1.00/2 = 32.50 \text{ in.}$$

$$d_{heel} = 42 - 12 - 1.00/2 = 29.50 \text{ in}$$

The depth of the compression block is:

$$a = \frac{A_s \cdot f_y}{(0.85 \cdot f'_c \cdot b)} = \frac{0.79 \cdot 60}{0.85 \cdot 4 \cdot 12} = 1.16 \text{ in}$$

The effective shear depth is:

$$d_{v,toe} = d - \frac{a}{2} = 32.50 - \frac{1.16}{2} = 31.92 \text{ in}$$

$$d_{v,heel} = d - a/2 = 29.50 - 1.16/2 = 28.92 \text{ in}$$

$d_v$  need be no less than  $0.9d_e$  :

For the toe,  $0.9 \cdot d_e = 0.9 \cdot d_{toe} = 0.9 \cdot 32.50 = 29.25 \text{ in}$

For the heel,  $0.9 \cdot d_e = 0.9 \cdot d_{heel} = 0.9 \cdot 29.50 = 26.55 \text{ in}$

Use  $d_{v,toe} = 31.92 \text{ in}$  and  $d_{v,heel} = 28.92 \text{ in}$

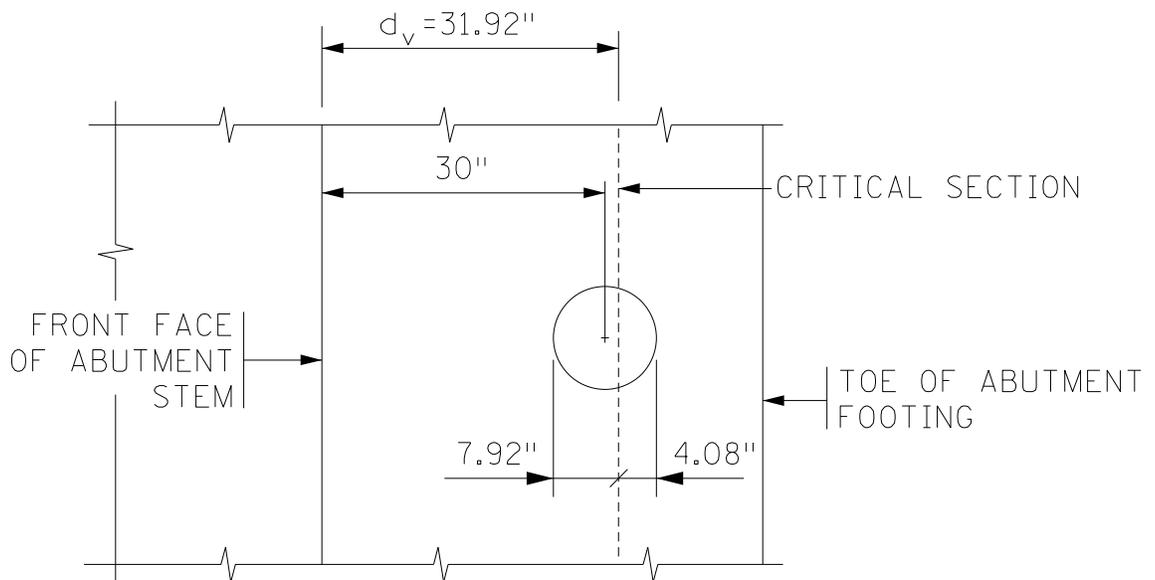
**[5.13.3.6.1]**  
**[5.8.3.2]**

**Check One-Way Shear in Footing**

The critical section is located  $d_v$  from the face of the abutment. The center line of the Row III piles is 54 inches from the back face of abutment. Therefore, the entire load from the Row III piles contributes to shear on the critical section. Ignore the beneficial effects of the vertical earth loads and footing self weight:

$$V_{u,Row III} = \text{Pile Reaction/Pile Spacing} = 178.0/8 = 22.3 \text{ kips/ft width}$$

The center line of the Row I piles is 30 inches from the front face of abutment. Therefore, only a portion of the load from the Row I piles contributes to shear on the critical section. See Figure 11.4.1.4.



**Figure 11.4.1.4**  
**Partial Footing Plan**

$$V_{u, \text{Row I}} = (\text{Pile Fraction Outside Critical Section}) \left( \frac{\text{Pile Reaction}}{\text{Pile Spacing}} \right)$$

$$V_{u, \text{Row I}} = (4.08/12) \cdot (199.7/8.00) = 8.5 \text{ kips/ft width}$$

The shear due to the Row III piles governs.

### [5.8.3.3]

There is no shear reinforcement, so the nominal shear capacity of the footing is:

$$V_n = V_c$$

An upper limit is placed on the maximum nominal shear capacity a section can carry. The maximum design shear for the footing heel is:

$$V_n = 0.25 \cdot f'_c \cdot b_v \cdot d_{v, \text{heel}} = 0.25 \cdot 4.0 \cdot 12.0 \cdot 28.92 = 347.0 \text{ kips}$$

The concrete shear capacity of a section is:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

### [5.8.3.4.1]

The distance from the point of zero shear to the backface of the abutment  $x_{v0}$  is:

$$x_{v0} = 54.0 + 6.0 = 60.0 \text{ in}$$

$$3 \cdot d_v = 3 \cdot 28.92 = 86.8 \text{ in} > 60.0 \text{ in}$$

Therefore,  $\beta = 2.0$

For a 1 ft. wide section, substituting values into  $V_c$  equation produces:

$$V_c = 0.0316 \cdot 2.0 \cdot \sqrt{4} \cdot 12 \cdot 28.92 = 43.9 \text{ kips}$$

This results in:

$$V_n = V_c = 43.9 \text{ kips} < 347.0 \text{ kips} \quad \text{OK}$$

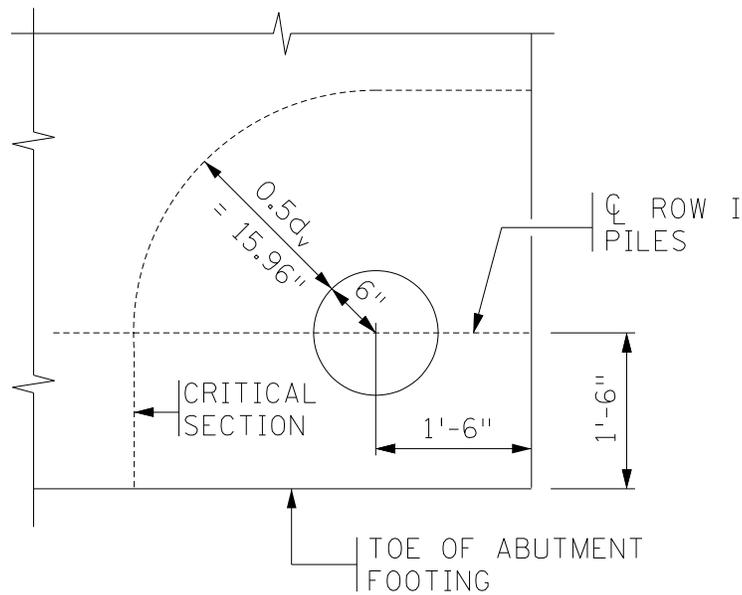
Including the shear resistance factor, the shear capacity is found to be:

$$V_r = \phi V_n = 0.90 \cdot 43.9 = 39.5 \text{ kips} > 22.3 \text{ kips} \quad \text{OK}$$

[5.13.3.6.1]

**Check Two-Way Shear in Footing**

Punching of an individual pile through the abutment footing is checked next. The critical section for two-way shear is located at  $0.5d_v$  from the perimeter of the pile. The Row I pile at the corner is the governing case because it has the largest load with the shortest length of critical section. See Figure 11.4.1.5.



**Figure 11.4.1.5  
Partial Footing Plan**

The length of the critical section is:

$$b_o = 18 + 0.5 \cdot \pi \cdot (15.96 + 6) + 18 = 70.5 \text{ in}$$

[5.13.3.6.3]

$$\begin{aligned} \phi V_n &= \phi (0.126 \cdot b_o \cdot d_{v, \text{toe}}) \\ &= 0.90(0.126)(70.5)(31.92) \\ &= 255.2 \text{ kips} \end{aligned}$$

$$V_u = \text{Row I Factored Pile Load} = 199.7 \text{ kips} < 255.2 \text{ kips} \quad \text{OK}$$

### I. Design Footing Reinforcement

The critical section for flexure in the footing is located at the face of the stem for both the top and bottom transverse reinforcement.

#### 1. Top Transverse Reinforcement Design For Strength Limit State

The factored moment,  $M_u$ , for the top transverse bars is found by assuming the heel acts as a cantilever supporting its self weight and the weight of the earth resting on it. In cases where the required reinforcement to resist these loads seems excessive, the moment due to the minimum back pile reaction may be included to decrease the top mat factored moment. Use the maximum load factors for DC and EV.

The distributed load associated with the self weight of the footing heel is:  
 $w_{ftg} = \gamma \cdot (\text{thickness}) \cdot (\text{width}) = 0.150 \cdot 3.5 \cdot 1.0 = 0.53 \text{ kips/ft}$

A heel length of 5.75 feet produces a moment of:

$$M_{DC} = w_{ftg} \cdot L \cdot \frac{L}{2} = 0.53 \cdot \frac{5.75^2}{2} = 8.8 \text{ kip - ft}$$

The distributed load associated with fill on top of the footing heel is:

$$w_{EV} = 0.120 \cdot (15.75 + 5.75) \cdot 1.0 = 2.58 \text{ kips/ft}$$

The associated moment in the footing at the stem is:

$$M_{EV} = 2.58 \cdot \frac{5.75^2}{2} = 42.7 \text{ kip - ft}$$

Combining loads to determine the design moment produces:

$$M_u = 1.25 \cdot M_{DC} + 1.35 \cdot M_{EV} = 1.25 \cdot 8.8 + 1.35 \cdot 42.7 = 68.6 \text{ kip - ft}$$

Determine the depth of the flexural reinforcement (assume #8 bars):

$$d = (\text{thickness}) - (\text{cover}) - \left(\frac{d_b}{2}\right) = 42 - 3 - \frac{1.00}{2} = 38.50 \text{ in}$$

#### [5.7.3.2]

Solve for the required area of reinforcing steel:

$$M_r = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left[ d - \frac{A_s \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b} \right] \geq M_u$$

Then for  $f'_c = 4.0 \text{ ksi}$  and assuming that  $\phi = 0.90$ ,

$$M_u = 0.90 \cdot A_s \cdot 60 \cdot \left[ d - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 12} \right] \cdot \frac{1}{12}$$

which can be rearranged to:

$$3.309 \cdot A_s^2 - 4.5 \cdot d \cdot A_s + M_u = 0$$

The required area of steel can be found by solving for the smaller root in the quadratic equation.

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618}$$

Then required area of steel is:

$$A_s = \frac{4.5 \cdot 38.50 - \sqrt{20.25 \cdot 38.50^2 - 13.236 \cdot 68.6}}{6.618} = 0.40 \text{ in}^2 / \text{ft}$$

Try #6 bars at 12 inches ( $A_s = 0.44 \text{ in}^2/\text{ft}$ ).

**[5.5.4.2.1]**

Check that assumed  $\phi = 0.90$  is correct.

$$\text{For \#6 bars, } d = 42 - 3 - \frac{0.75}{2} = 38.63 \text{ in}$$

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{0.44 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 0.76 \text{ in}$$

**[5.7.2.1]**

Concrete compression strain limit  $\epsilon_c = 0.003$

**[Table C5.7.2.1-1]**

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (38.63 - 0.76) \left( \frac{0.003}{0.76} \right) = 0.149 > \epsilon_{tl} = 0.005$$

Therefore,  $\phi = 0.90$  OK

Try #6 bars at 12 inch spacing ( $A_s = 0.44 \text{ in}^2/\text{ft}$ ).

**[5.7.3.3.2]**

**Check Minimum Reinforcement**

The minimum reinforcement check is the amount of flexural reinforcement needed to carry the lesser of the cracking moment or 1.33 times the original design moment.

The concrete density modification factor,  $\lambda$ , for normal weight concrete is 1.0.

The rupture stress of concrete in flexure is:

**[5.4.2.6]**

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{4} = 0.48 \text{ ksi}$$

The section modulus is:

$$S = \frac{1}{6} \cdot b \cdot t^2 = \frac{1}{6} \cdot 12 \cdot (42)^2 = 3528 \text{ in}^3$$

Take  $\gamma_1 = 1.60$  and  $\gamma_3 = 0.67$  for ASTM A615 Grade 60 reinforcement.

Combining these parameters leads to a cracking moment of:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S = 0.67 \cdot 1.6 \cdot 0.48 \cdot 3528 \cdot \frac{1}{12} = 151.3 \text{ k-ft}$$

The other criterion is:

$$1.33 \cdot M_u = 1.33 \cdot 68.6 = 91.2 \text{ kip-ft} \quad \underline{\text{GOVERNS}}$$

The capacity of the #6 bars at a 12 inch spacing is:

$$M_r = \phi \cdot A_s \cdot f_y \cdot \left( d - \frac{a}{2} \right)$$

$$M_r = 0.9 \cdot 0.44 \cdot 60 \cdot \left( 38.63 - \frac{0.76 \cdot 0.85}{2} \right) \cdot \left( \frac{1}{12} \right) = 75.8 \text{ kip-ft} < 91.2 \text{ kip-ft}$$

Try #7 bars at 12 inch spacing ( $A_s = 0.60 \text{ in}^2$ ):

$$d = 38.56 \text{ in}$$

$$a = 0.88 \text{ in} \quad c = 1.04 \text{ in}$$

$$\epsilon_t = 0.108 > \epsilon_{ti} = 0.005 \quad \underline{\text{OK}}$$

$$M_r = 102.9 \text{ kip-ft} > 91.2 \text{ kip-ft} \quad \underline{\text{OK}}$$

Provide #7 bars at 12 inch spacing ( $A_s = 0.60 \text{ in}^2$ )

## 2. Bottom Transverse Reinforcement

### Design For Strength Limit State

Although the toe has a greater thickness than the heel, for simplicity assume a constant thickness of 42 inches. Then the factored moment for the bottom mat is the largest of the moments due to the maximum pile reactions for the Row I or Row III piles.

For the Row I piles:

$$\begin{aligned} M_{u\text{RowI}} &= \left( \frac{\text{Pile Reaction}}{\text{Pile Spacing}} \right) (\text{Moment Arm}) \\ &= \left( \frac{199.7}{8.0} \right) (4.00 - 1.50) = 62.4 \text{ kip-ft/ft width} \end{aligned}$$

For the Row III piles, subtract off the moment due to earth on the heel (see earlier calculation for  $M_{EV}$ ) when calculating the factored moment. (Use minimum load factor for EV,  $\gamma = 1.0$ ):

$$M_{uRowIII} = \left( \frac{\text{Pile Reaction}}{\text{Pile Spacing}} \right) (\text{Moment Arm}) - \gamma \cdot M_{EV}$$

$$= \left( \frac{178.0}{8.0} \right) (5.75 - 1.25) - 1.0 (42.7) = 57.4 \text{ kip-ft/ft width}$$

The Row I moment governs.  $M_{udes} = 62.4$  kip-ft/ft width

Assuming #8 bars, the depth of the bottom flexural reinforcement is:

$$d = (\text{thickness}) - (\text{pile embedment}) - \left( \frac{d_b}{2} \right) = 42 - 12 - \frac{1.00}{2} = 29.50 \text{ in}$$

Solve once again with:

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618}$$

$$A_s = \frac{4.5 \cdot 29.50 - \sqrt{20.25 \cdot 29.50^2 - 13.236 \cdot 62.4}}{6.618} = 0.48 \text{ in}^2 / \text{ft}$$

The required area of steel is 0.48 in<sup>2</sup>/ft. Try #7 bars at 12 inches with standard hooks ( $A_s = 0.60$  in<sup>2</sup>/ft).

#### [5.5.4.2.1]

Check that assumed  $\phi = 0.90$  is correct:

$$\text{For \#7 bars, } d = 42 - 12 - \frac{0.875}{2} = 29.56 \text{ in}$$

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{0.60 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 1.04 \text{ in}$$

#### [5.7.2.1]

Concrete compression strain limit  $\epsilon_c = 0.003$

#### [Table C5.7.2.1-1]

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \cdot \left( \frac{\epsilon_c}{c} \right) = (29.56 - 1.04) \cdot \left( \frac{0.003}{1.04} \right) = 0.082 > \epsilon_{tl} = 0.005$$

Therefore,  $\phi = 0.90$  OK

#### [5.7.3.3.2]

#### Check Minimum Reinforcement

The minimum reinforcement check for the bottom of the footing has the same steps as the other elements.

Using the simplified constant thickness of 42 inches, previous calculations result in a value for  $M_{cr}$  of:

$$M_{Cr} = 151.3 \text{ kip} \cdot \text{ft}$$

The other criterion is:

$$1.33 \cdot M_u = 1.33 \cdot 62.4 = 83.0 \text{ kip} \cdot \text{ft} \quad \underline{\text{GOVERNS}}$$

The capacity of the #7 bars at a 12 inch spacing is:

$$M_r = \phi \cdot A_s \cdot f_y \cdot \left( d - \frac{a}{2} \right)$$

$$M_r = 0.9 \cdot 0.60 \cdot 60 \cdot \left( 29.56 - \frac{1.04 \cdot 0.85}{2} \right) \cdot \left( \frac{1}{12} \right)$$

$$= 78.6 \text{ kip} \cdot \text{ft} < 83.0 \text{ kip} \cdot \text{ft} \quad \underline{\text{NO GOOD}}$$

Revise reinforcement to #8 bars at 12 inches ( $A_s = 0.79 \text{ in}^2/\text{ft}$ ).

Then:

$$d = 29.50 \text{ in}$$

$$a = 1.16 \text{ in} \quad c = 1.36 \text{ in}$$

$$\epsilon_t = 0.062 > \epsilon_{tl} = 0.005 \quad \underline{\text{OK}}$$

$$M_r = 102.8 \text{ kip} \cdot \text{ft} > 83.0 \text{ kip} \cdot \text{ft} \quad \underline{\text{OK}}$$

Provide #8 bars at 12 inch spacing ( $A_s = 0.79 \text{ in}^2$ )

### 3. Longitudinal Reinforcement

#### Design For Strength Limit State

For longitudinal bars, design for uniform load due to all vertical loads spread equally over the length of the footing. Assume the footing acts as a continuous beam over pile supports. Use the longest pile spacing for design span.

Then based on the maximum vertical load from Table 11.4.1.5:

$$w_u = \frac{4273}{59.00} = 72.4 \text{ kips/ft}$$

$$M_u = \frac{w_u L^2}{10} = \frac{72.4 \cdot (8.0)^2}{10} = 463.4 \text{ kip} \cdot \text{ft}$$

Assume #6 bars, which is the smallest size used by MnDOT in footings:

$$d = 42 - 12 - 1.00 - \frac{0.75}{2} = 28.63 \text{ in}$$

Assuming  $\phi = 0.90$ , solve for required area of reinforcement:

$$M_r = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left( d - \frac{A_s \cdot f_y}{2 \cdot 0.85 \cdot f_c \cdot b} \right) \geq M_u$$

Then:

$$463.4 = 0.90 \cdot A_s \cdot 60 \cdot \left( 28.63 - \frac{A_s \cdot 60}{2 \cdot 0.85 \cdot 4 \cdot 171} \right) \cdot \frac{1}{12}$$

Rearrange and get  $0.2322 \cdot A_s^2 - 128.84 \cdot A_s + 463.4 = 0$

Solving, minimum  $A_s = 3.62 \text{ in}^2$

Try 11-#6 bars. ( $A_s = 4.84 \text{ in}^2$ )

Check that assumed  $\phi = 0.90$  is correct:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{4.84 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 171} = 0.59 \text{ in}$$

**[5.7.2.1]**

Concrete compression strain limit  $\epsilon_c = 0.003$

**[Table C5.7.2.1-1]**

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \cdot \left( \frac{\epsilon_c}{c} \right) = (28.63 - 0.59) \cdot \left( \frac{0.003}{0.59} \right) = 0.143 > \epsilon_{tl} = 0.005$$

Therefore  $\phi = 0.90$

OK

$M_r = 618.1 \text{ kip-ft}$

**Check Minimum Reinforcement**

The rupture stress of concrete in flexure was previously calculated as:

**[5.4.2.6]**

$$f_r = 0.48 \text{ ksi}$$

The section modulus is:

$$S = \frac{1}{6} \cdot b \cdot t^2 = \frac{1}{6} \cdot 171 \cdot (42)^2 = 50,274 \text{ in}^3$$

Take  $\gamma_1 = 1.60$  and  $\gamma_3 = 0.67$  for ASTM A615 Grade 60 reinforcement.

Combining these parameters leads to a cracking moment of:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S = 0.67 \cdot 1.6 \cdot 0.48 \cdot 50,274 \cdot \frac{1}{12} = 2155.7 \text{ kip-ft}$$

The other criterion is:

$$1.33 \cdot M_u = 1.33 \cdot 463.4 = 616.3 \text{ kip-ft} \quad \underline{\text{GOVERNS}}$$

$M_r = 618.1 \text{ kip-ft} > 616.3 \text{ kip-ft}$

OK

Provide 11-#6 bars ( $A_s = 4.84 \text{ in}^2$ ), top and bottom, for the footing longitudinal reinforcement.

**J. Flexural Design of the Stem**

The moments associated with the eccentricity of vertical loads are minimal and are therefore ignored. Use a one-foot wide design strip.

The stem design is governed by the horizontal earth pressure and live load surcharge loading during construction.

**[3.11.5.5]****Horizontal Earth Pressure**

$$p_{\text{top}} = 0.0 \text{ ksf}$$

$$p_{\text{bottom}} = 0.033 \cdot 21.50 = 0.710 \text{ ksf}$$

The resultant force applied to the stem is:

$$P_{\text{EH}} = 0.5 \cdot (0.710) \cdot (21.50) \cdot (1.00) = 7.63 \text{ kips}$$

The height of the resultant above the footing is:

$$x_{\text{EH}} = \frac{21.50}{3} = 7.17 \text{ ft}$$

The moment at the base of the stem is:

$$M_{\text{EH}} = P_{\text{EH}} \cdot x_{\text{EH}} = 7.63 \cdot 7.17 = 54.7 \text{ kip-ft}$$

**[Table 3.11.6.4-1]****Live Load Surcharge**

For walls over 20 feet in height,  $h_{\text{eq}}$  is 2 feet.

The resultant force applied to the stem is:

$$P_{\text{LS}} = 0.033 \cdot (2.00) \cdot (21.50) \cdot (1.00) = 1.42 \text{ kips}$$

The height of the resultant force above the footing is:

$$x_{\text{LS}} = \frac{21.50}{2} = 10.75 \text{ ft}$$

The moment at the base of the stem is:

$$M_{\text{LS}} = P_{\text{LS}} \cdot x_{\text{LS}} = 1.42 \cdot 10.75 = 15.3 \text{ kip-ft}$$

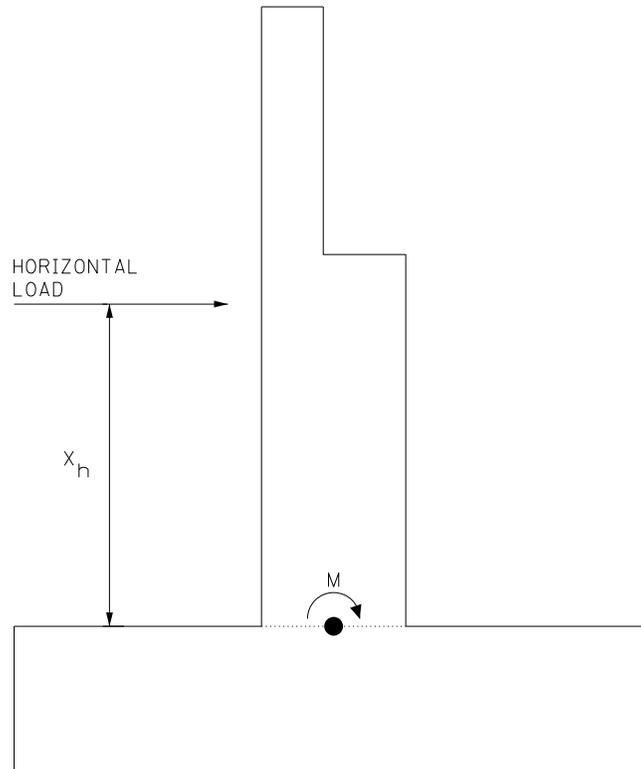
**Design Moments**

The design factored moment is:

$$M_u = 1.5 \cdot M_{\text{EH}} + 1.75 \cdot M_{\text{LS}} = 1.50 \cdot 54.7 + 1.75 \cdot 15.3 = 108.8 \text{ kip-ft}$$

The design service moment is:

$$M_{\text{service}} = 1.0 \cdot M_{\text{EH}} + 1.0 \cdot M_{\text{LS}} = 1.0 \cdot 54.7 + 1.0 \cdot 15.3 = 70.0 \text{ kip-ft}$$



**Figure 11.4.1.6**  
**Load Diagram for Stem Design**

[5.7.2.2]

[5.7.3.2]

**Investigate the Strength Limit State**

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment.

$$M_u = 108.8 \text{ kip - ft}$$

Initially, assume that #6 bars are used for flexural reinforcement to compute the "d" dimension:

$$d = (\text{thickness}) - (\text{cover}) - \left(\frac{d_b}{2}\right) = 54 - 2 - \frac{0.75}{2} = 51.63 \text{ in}$$

For  $f'_c = 4.0 \text{ ksi}$  and assuming  $\phi = 0.90$ , it was shown earlier that:

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618}$$

Then required area of steel is  $0.47 \text{ in}^2/\text{ft}$ .

Try #7 bars at 12 inches ( $A_s=0.60 \text{ in}^2/\text{ft}$ )

$$d = 51.56 \text{ in}$$

$$a = 0.88 \text{ in}$$

$$M_r = 138.0 \text{ kip-ft}$$

Check that assumed  $\phi = 0.90$  is correct.

$$c = a/\beta_1 = 0.88/0.85 = 1.04 \text{ in}$$

[5.7.2.1]

Concrete compression strain limit  $\epsilon_c = 0.003$

[Table C5.7.2.1-1]

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (51.56 - 1.04) \left( \frac{0.003}{1.04} \right) = 0.146 > \epsilon_{tl} = 0.005$$

Therefore  $\phi = 0.90$

OK

[5.7.3.4]

**Crack Control**

Check crack control equations to ensure that the primary reinforcement is well distributed.

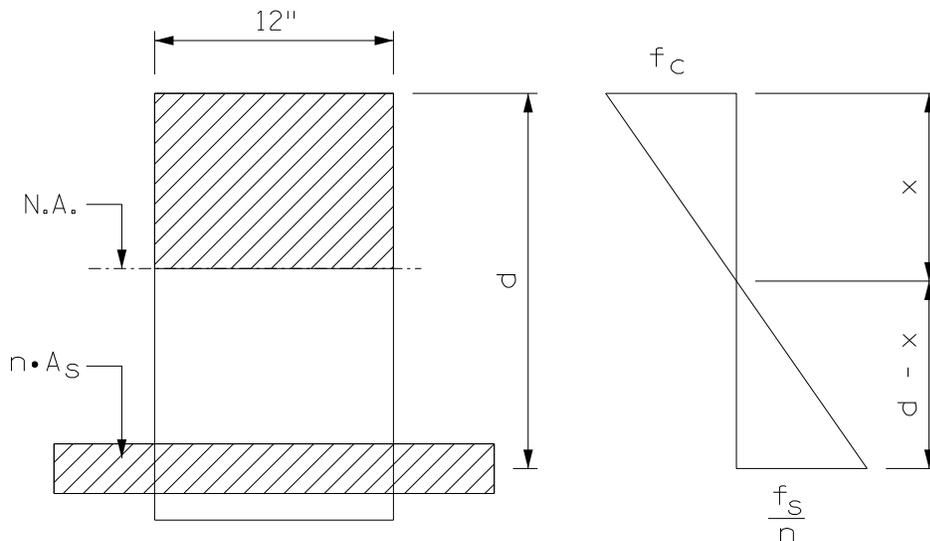
Compute the modular ratio for 4.0 ksi concrete:

$$n = \frac{E_s}{E_c} = \frac{29,000}{33,000 \cdot (0.145)^{1.5} \cdot \sqrt{4}} = 7.96$$

Use 8

The transformed area of the reinforcement is:

$$n \cdot A_s = 8 \cdot 0.60 = 4.80 \text{ in}^2$$



**Figure 11.4.1.7**

Determine the location of the neutral axis:

$$\frac{1}{2} \cdot b x^2 = n \cdot A_s (d - x)$$

$$\frac{1}{2} \cdot (12) \cdot x^2 = 4.80 (51.56 - x) \quad \text{solving, } x = 6.03 \text{ inches}$$

$$j \cdot d = d - \frac{x}{3} = 51.56 - \frac{6.03}{3} = 49.55 \text{ in}$$

$$\text{Actual } f_s = \frac{M_{\text{service}}}{A_s \cdot j \cdot d} = \frac{70.0 \cdot 12}{0.60 \cdot (49.55)} = 28.3 \text{ ksi}$$

Concrete cover = 2 in

$$d_c = \text{concrete cover} + \frac{d_b}{2} = 2 + \frac{0.875}{2} = 2.44 \text{ in}$$

For Class 1 exposure,  $\gamma_e = 1.0$  and  $h = 54$  in:

$$\beta_s = 1 + \frac{d_c}{0.7 (h - d_c)} = 1 + \frac{2.44}{0.7 (54 - 2.44)} = 1.068$$

$$\text{Allowable } f_s = \frac{700 \gamma_e}{\beta_s \cdot (s + 2 \cdot d_c)} = \frac{700 \cdot 1.0}{1.068 \cdot (12 + 2 \cdot 2.44)}$$

$$= 38.8 \text{ ksi, but must be } \leq 0.6f_y = 36.0 \text{ ksi}$$

Allowable  $f_s = 36.0 \text{ ksi} > 28.3 \text{ ksi}$

OK

### [5.7.3.3.2]

#### Check Minimum Reinforcement

The factored flexural resistance must be greater than the lesser of  $M_{cr}$  and  $1.33 \cdot M_u$ .

The section modulus is:

$$S = \frac{1}{6} \cdot b \cdot t^2 = \frac{1}{6} \cdot 12 \cdot (54)^2 = 5832 \text{ in}^3$$

$$\gamma_1 = 1.60 \text{ (for other concrete structures)}$$

$$\gamma_3 = 0.67 \text{ (for ASTM A615 Grade 60 reinforcement).}$$

Combining these parameters and using the rupture stress computed earlier leads to a cracking moment of:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S = 0.67 \cdot 1.6 \cdot \frac{0.48 \cdot 5832}{12} = 250.1 \text{ kip-ft}$$

The factored flexural resistance must be greater than the lesser of  $M_{cr}$  or  $1.33M_u$ .

$$1.33 \cdot M_u = 1.33 \cdot 108.8 = 144.7 \text{ kip-ft}$$

$$\text{Actual } M_r = 138.0 \text{ k-ft} < 144.7 \text{ kip-ft}$$

GOVERNS  
NO GOOD

Try #6 bars at 6 inch spacing ( $A_s=0.88 \text{ in}^2/\text{ft}$ ):

$$d = 51.63 \text{ in}$$

$$a = 1.29 \text{ in} \quad c = 1.52 \text{ in}$$

$$\epsilon_t = 0.099 > \epsilon_{ti} = 0.005$$

OK

$$M_r = 201.9 \text{ kip-ft}$$

Provide #6 bars at 6 inches ( $A_s=0.88 \text{ in}^2/\text{ft}$ ) for vertical back face dowels.

### Splice Length

#### [5.11.2.1.1]

Calculate the tension lap length for the stem vertical reinforcing. For epoxy coated #6 bars the basic development length  $\ell_{db}$  is:

$$\ell_{db} = \frac{2.4 \cdot d_b \cdot f_y}{\sqrt{f'_c}} = \frac{2.4 \cdot 0.75 \cdot 60}{\sqrt{4.0}} = 54.0 \text{ in.}$$

The modification factors to the development length are:

$\lambda_{cf} = 1.5$  for epoxy coated bars with cover less than three bar diameters (2.25 in).

$\lambda_{rl} = 1.0$  for vertical bars

$\lambda = 1.0$  for normal weight concrete

$\lambda_{er} = 1.0$  taken conservatively assuming  $A_{s\text{provided}} = A_{s\text{required}}$

For determination of  $\lambda_{rc}$ :

$c_b = 2.38 \text{ in.}$  (governed by  $2.0 \text{ clear} + 0.5 \cdot \text{bar diameter}$ )

$A_{tr} = 0$  since there are no bars that cross the potential splitting planes

Then  $k_{tr} = 0$

$$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}} = \frac{0.75}{2.38 + 0} = 0.32 < 0.4$$

So  $\lambda_{rc} = 0.4$

Then the development length  $\ell_d$  is:

$$\ell_d = \frac{\ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = \frac{54.0 \cdot (1.0 \cdot 1.5 \cdot 0.4 \cdot 1.0)}{1.0} = 32.40 \text{ in.}$$

#### [5.11.5.3.1]

Use a Class B splice.

The required lap length  $\ell_{spl}$  is:

$$\ell_{spl} = 1.3 \cdot \ell_d = 1.3 \cdot 32.40 = 42.12 \text{ in}$$

Therefore, the tension lap length must be at least 3'-7".

To produce an efficient design, determine the transition point above the footing where the reinforcement can be changed to #6 bars at 12 inches.

The stem's factored flexural resistance utilizing #6 bars at 12 inches is:

$$M_r = \phi M_n = \phi A_s \cdot f_s \cdot \left( d - \frac{a}{2} \right)$$

$$a = \frac{A_s \cdot f_s}{0.85 \cdot f'_c \cdot b} = \frac{0.44 \cdot 60}{0.85 \cdot 4.0 \cdot 12} = 0.65 \text{ in}$$

$$M_r = 0.9 \cdot 0.44 \cdot 60 \cdot \left( 51.63 - \frac{0.65}{2} \right) \cdot \frac{1}{12} = 101.6 \text{ kip - ft}$$

The  $1.33M_u$  criteria will control, so the maximum factored moment at the transition point  $M_{utrans}$  can be determined as follows:

$$M_r = 1.33 \cdot M_{utrans}$$

$$M_{utrans} = \frac{M_r}{1.33} = \frac{101.6}{1.33} = 76.4 \text{ k - ft}$$

The depth that this factored moment occurs can be determined from the following:

$$M_{utrans} = 1.5 M_{EHtrans} + 1.75 M_{LStrans}$$

$$M_{utrans} = 1.5 \cdot \left( \frac{1}{2} \cdot 0.033 \cdot h_{trans}^2 \right) \cdot \left( \frac{h_{trans}}{3} \right) + 1.75 \cdot \frac{1}{2} \cdot 0.033 \cdot 2.0 \cdot h_{trans}^2$$

$$M_{utrans} = 0.00825h_{trans}^3 + 0.05775h_{trans}^2$$

Solving for  $h_{trans}$ , the maximum wall height at which #6 bars at 12 inches is adequate is 18.9 feet, say 18'-10".

Then, the height above the footing that #6 bars at 6 inches is required is:

$$21.5 - 18.83 = 2.67 \text{ ft}$$

The crack control requirements also need to be verified at this location.

The procedure above yields the following results:

Service moment at transition point,  $M_{strans} = 48.9$  kip-ft

Neutral axis location,  $x = 5.22$  in

$jd = 49.89$  in

Actual  $f_s = 26.7$  ksi < Permitted  $f_s = 36.0$  ksi in OK

In summary, provide #6 bars at 6 inches for the back face dowels that extend 2'-8" plus a lap length (3'-7") beyond the top of the footing. In addition, provide #6 bars at 12 inches for the full height of the stem.

### [5.10.8]

#### Shrinkage and Temperature Reinforcement

Reinforcement is required on the faces of the abutment stem to resist cracking due to shrinkage and temperature.

$$b = 15.75 \text{ ft} = 189 \text{ in}$$

$$h = 54 \text{ in}$$

$$A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.30 \cdot 189 \cdot 54}{2 \cdot (189 + 54) \cdot 60} = 0.46 \text{ in}^2/\text{ft}$$

Use #6 bars at 10 inches ( $A_s=0.53$  in<sup>2</sup>/ft) on each face, for the horizontal reinforcement and #6 bars at 10 inches for the vertical front face reinforcement. The required shrinkage and temperature reinforcement is 4.5% greater than the #6 bars at 12 inches ( $A_s = 0.44$  in<sup>2</sup>/ft) previously determined for the back face verticals, so some adjustments are necessary. Revise the previously designed back face vertical bars to #6 bars at 10 inches and the previously designed back face dowels to #6 bars at 5 inches.

#### L. Flexural Design of the Backwall (Parapet)

The required vertical reinforcement in the backwall (parapet) is sized to carry the moment at the bottom of the backwall. The design is performed on a one-foot wide strip of wall. The backwall design is governed by the horizontal earth pressure and live load surcharge loading during construction.

#### Horizontal Earth Pressure

$$p_{top} = 0.0 \text{ ksf}$$

$$p_{bottom} = 0.033 \cdot 5.75 = 0.190 \text{ ksf}$$

The resultant force applied to the backwall is:

$$P_{EH} = 0.5 \cdot (0.190) \cdot (5.75) \cdot (1.00) = 0.55 \text{ kips}$$

The height of the resultant above the bottom of the backwall is:

$$x_{EH} = \frac{(5.75)}{3} = 1.92 \text{ ft}$$

The moment at the bottom of the backwall is:

$$M_{EH} = P_{EH} \cdot x_{EH} = 0.55 \cdot 1.92 = 1.06 \text{ kip - ft}$$

**[Table 3.11.6.4-1]**

**Live Load Surcharge**

Interpolate between the values provided in the table to arrive at the required equivalent height of surcharge to use for the design of the backwall.

$$h_{eq} = \left( \frac{5.75 - 5}{10.0 - 5} \right) \cdot (3 - 4) + 4 = 3.85 \text{ ft}$$

The resultant force applied to the backwall is:

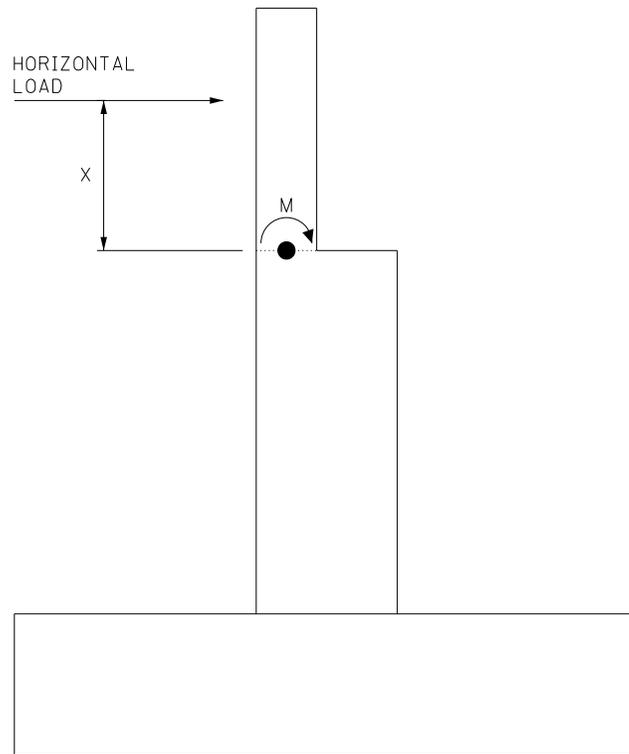
$$P_{LS} = 0.033 \cdot (3.85) \cdot (5.75) \cdot (1.00) = 0.73 \text{ kips}$$

The height of the resultant force above the bottom of the backwall is:

$$x_{LS} = \frac{5.75}{2} = 2.88 \text{ ft}$$

Moment at the bottom of the backwall is:

$$M_{LS} = P_{LS} \cdot x_{LS} = 0.73 \cdot 2.88 = 2.10 \text{ kip - ft}$$



**Figure 11.4.1.8**  
**Load Diagram for Backwall Design**

**Design Moments**

Combining the load factors for the EH and LS load components with the flexural design forces at the bottom of the backwall produces the following design forces.

$$M_U = 1.5M_{EH} + 1.75M_{LS} = 1.5(1.06) + 1.75(2.10) = 5.27 \text{ kip - ft}$$

$$M_{service} = M_{EH} + M_{LS} = 1.06 + 2.10 = 3.16 \text{ kip - ft}$$

**[5.7.2.2]**

**[5.7.3.2]**

**Investigate the Strength Limit State**

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment.

Once again, for  $f'_c = 4 \text{ ksi}$  and assuming  $\phi = 0.90$ :

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_U}}{6.618}$$

Initially, assume that #6 bars are used for flexural reinforcement to compute the "d" dimension:

$$d = (\text{thickness}) - (\text{clear cover}) - \left(\frac{d_b}{2}\right) = 18 - 2 - \frac{0.75}{2} = 15.63 \text{ in}$$

Solving the equation, the required area of steel is 0.075 in<sup>2</sup>/ft.

In order to match the spacing of the stem reinforcement, try #5 bars at a 10 inch spacing. ( $A_s = 0.37 \text{ in}^2/\text{ft}$ ,  $d = 15.69 \text{ in}$ ,  $M_r = 25.7 \text{ k-ft}$ ).

Check that assumed  $\phi = 0.90$  is correct.

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{0.37 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 0.64 \text{ in.}$$

**[5.7.2.1]**

Concrete compression strain limit  $\varepsilon_c = 0.003$

**[Table C5.7.2.1-1]**

Reinforcement tension-controlled strain limit  $\varepsilon_{tl} = 0.005$

$$\varepsilon_t = (d - c) \left(\frac{\varepsilon_c}{c}\right) = (15.69 - 0.64) \left(\frac{0.003}{0.64}\right) = 0.071 > \varepsilon_{tl} = 0.005$$

Therefore  $\phi = 0.90$  OK

**[5.7.3.4]**

**Check Crack Control**

Check crack control equations to ensure that the primary reinforcement is well distributed.

The transformed area of the reinforcement is:

$$n \cdot A_s = 8 \cdot 0.37 = 2.96 \text{ in}^2$$

Determine the location of the neutral axis:

$$\frac{1}{2} \cdot b x^2 = n \cdot A_s (d - x)$$

$$\frac{1}{2} \cdot (12) \cdot x^2 = 2.96 (15.69 - x) \quad \text{solving, } x = 2.55 \text{ inches}$$

$$j \cdot d = d - \frac{x}{3} = 15.69 - \frac{2.55}{3} = 14.84 \text{ in}$$

$$\text{Actual } f_{ss} = \frac{M_{\text{service}}}{A_s \cdot j \cdot d} = \frac{3.16 \cdot 12}{0.37 \cdot (14.84)} = 6.9 \text{ ksi}$$

$$d_c = \text{concrete cover} + \frac{d_b}{2} = 2 + \frac{0.625}{2} = 2.31 \text{ in}$$

For Class 1 exposure ( $\gamma_e=1.0$ ), and  $h=18$  in:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{2.31}{0.7(18 - 2.31)} = 1.21$$

$$\text{Then allowable } f_{ssa} = \frac{700 \gamma_e}{\beta_s \cdot (s + 2 \cdot d_c)} < 0.6 \cdot f_y = 36 \text{ ksi}$$

$$= \frac{700 \cdot 1.0}{1.21 \cdot (12 + 2 \cdot 2.31)} = 34.8 \text{ ksi} > 6.9 \text{ ksi} \quad \underline{\text{OK}}$$

### [5.7.3.3.2]

#### Check Minimum Reinforcement

The section modulus is:

$$S = \frac{1}{6} \cdot b \cdot t^2 = \frac{1}{6} \cdot 12 \cdot (18)^2 = 648 \text{ in}^3$$

Taking  $\gamma_1 = 1.60$  and  $\gamma_3 = 0.67$  (for ASTM A615 Grade 60) and using the rupture stress computed earlier, the cracking moment is:

$$M_{cr} = \gamma_1 \gamma_3 \frac{f_r \cdot I_g}{Y_t} = 0.67 \cdot 1.6 \cdot \frac{0.48 \cdot 648}{12} = 27.8 \text{ kip-ft}$$

The factored flexural resistance must be greater than the lesser of  $M_{cr}$  or  $1.33M_u$ :

$$1.33 \cdot M_u = 1.33 \cdot 5.27 = 7.0 \text{ kip-ft} \quad \underline{\text{GOVERNS}}$$

$$\text{Actual } M_r = 25.7 \text{ kip-ft} > 7.0 \text{ kip-ft} \quad \underline{\text{OK}}$$

Use #5 bars at 10 inches for vertical back face reinforcement.

### [5.10.8]

#### Shrinkage and Temperature Reinforcement

To distribute and limit the size of cracks associated with concrete shrinkage and with temperature changes, a modest amount of reinforcement is provided transverse to the primary reinforcement.

$$b = 5.75 \text{ ft} = 69 \text{ in}$$

$$h = 18 \text{ in}$$

$$A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} = \frac{1.30 \cdot 69 \cdot 18}{2 \cdot (69+18) \cdot 60} = 0.15 \text{ in}^2/\text{ft}$$

Provide horizontal #5 bars at 12 inches to both faces,  $A_s = 0.31 \text{ in}^2/\text{ft}$

The final reinforcing layout is presented in Figure 11.4.1.9.

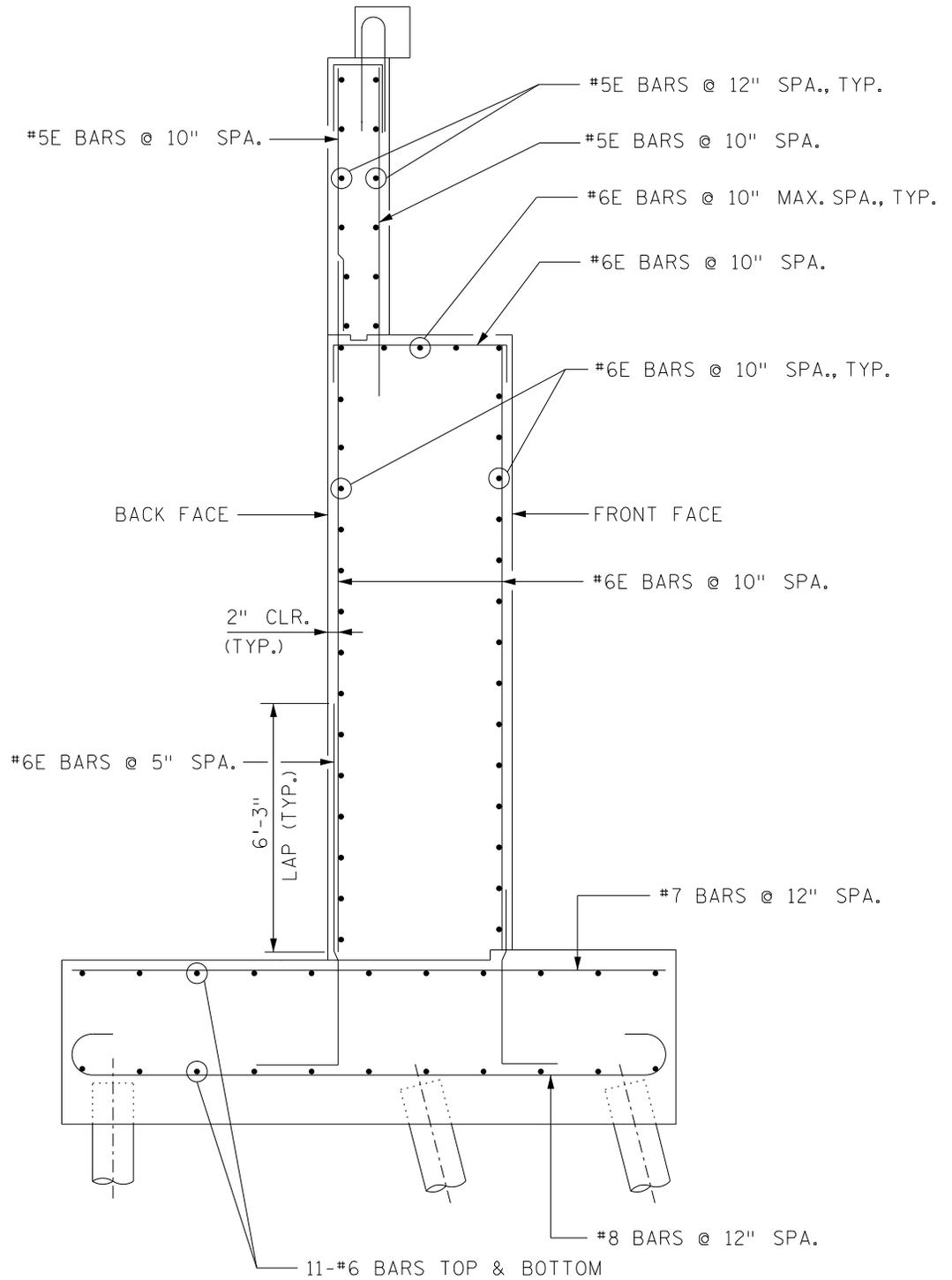


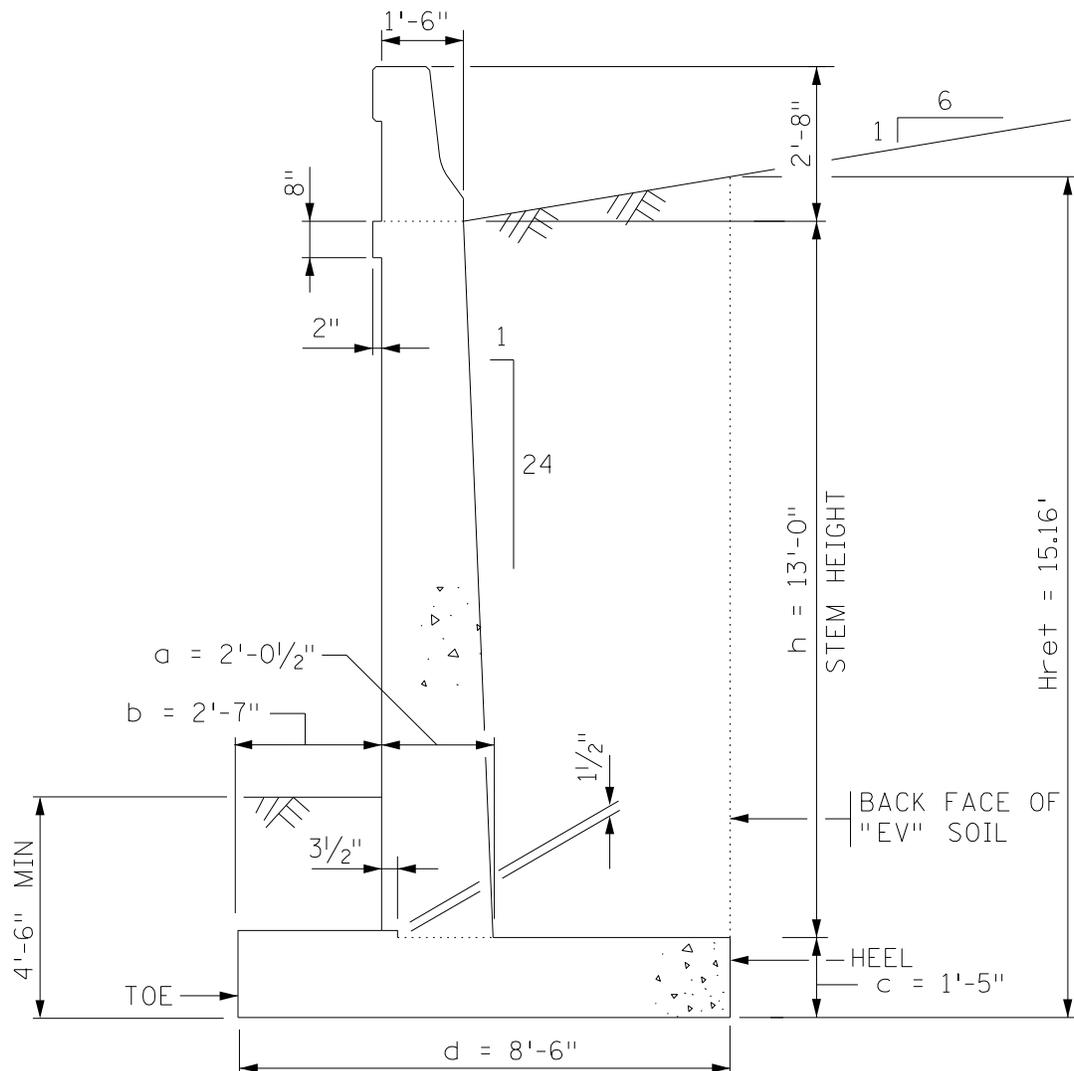
Figure 11.4.1.9

**[This page intentionally left blank.  
Wingwall design example to be added in the future.]**

**11.4.2 Retaining Wall Design Example**

This example illustrates the design of a cantilever retaining wall supported on a spread footing, details of which can be found in the MnDOT Standard Plan Sheets 5-297.620 to 635. The wall has a stem height of 13'-0" and supports an "F" rail, a 2'-0" live load surcharge, and a back slope that can vary from level to 1V:6H. After determining the load components and design loads, the global behavior of the retaining wall is evaluated. This includes: an eccentricity (overturning) check, a bearing stress check, and a sliding check, after which the wall section is designed.

The wall cross-section is shown in Figure 11.4.2.1. As a starting point, choose a footing width that is 60 to 70 percent of the stem height, and a footing thickness that is 10 to 15 percent of the stem height. Choose a toe projection that is approximately 30 percent of the footing width.



**Figure 11.4.2.1**

The current MnDOT LRFD Cast-In-Place Retaining Wall Standards (MnDOT CIP Wall Standards) were designed using the 2010 AASHTO LRFD code. This example is based on the current specifications and therefore, some of the requirements used will differ from the MnDOT CIP Wall Standards. For new designs that fall outside the limits of the MnDOT CIP Wall Standards, follow the current AASHTO requirements.

Material and design parameters used in this example are:

Soil:

The soil is noncohesive.

Unit weight of fill,  $\gamma_s = 0.120$  kcf

Retained soil friction angle,  $\phi_{\text{fret}} = 35^\circ$

Soil wall friction angle,  $\delta = (2/3) \cdot \phi_{\text{fret}} = (2/3) \cdot 35^\circ = 23.33^\circ$

Backfill slope (1V:6H) angle,  $\beta = 9.46^\circ$

Angle between back face of wall and horizontal,  $\theta = 90^\circ$

(Note that for semi-gravity cantilevered walls with heels, a failure surface along the back face of the wall would be interfered with by the heel. So for this type of wall, the failure surface becomes a plane extending vertically up from the end of the heel and the back face of the "EV" soil is considered the back face of wall.)

Internal friction angle of foundation soil,  $\phi_{\text{found}} = 32^\circ$

Concrete:

Strength at 28 days,  $f'_c = 4.0$  ksi

Unit weight,  $w_c = 0.150$  kcf

Reinforcement:

Yield strength,  $f_y = 60$  ksi

Modulus of elasticity,  $E_s = 29,000$  ksi

Barrier:

F-barrier weight = 0.464 k/ft

F-barrier centroid from outside barrier face = 0.53 ft

**[C3.11.5.3]**

### **A. Bearing Capacity**

This is a design example for a MnDOT standard wall, so the site specific conditions of where this retaining wall would be built are unknown. Therefore, the applied bearing pressures will be determined for this wall, but not checked against a maximum. The allowable bearing capacity for the specific wall location must be determined by a geotechnical engineer.

### **B. Loads**

The design of the retaining wall is performed on a 1'-0" wide strip. Figure 11.4.2.2 shows a section of the retaining wall. Soil and concrete elements are broken into rectangles or triangles. Each rectangle or triangle is labeled with two numbers. The first number is the unfactored load and the second number (in parentheses) is the horizontal distance

"x" or vertical distance "y" from the toe of the footing to the center of load application. Calculations are shown below for earth loads, live load surcharge, and barrier collision load. All of the loads are summarized in Tables 11.4.2.1 and 11.4.2.2.

**C. Earth Pressure  
(EH and EV)  
[3.11.5]**

Use the Coulomb theory of earth pressure to determine the magnitude of active earth pressure.

$$K_a = \frac{\sin^2(\theta + \phi_{\text{fret}})}{\sin^2(\theta)\sin(\theta - \delta) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi_{\text{fret}} + \delta) \cdot \sin(\phi_{\text{fret}} - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}} \right]^2}$$

$$K_a = \frac{\sin^2(90 + 35)}{\sin^2(90)\sin(90 - 23.33) \cdot \left[ 1 + \sqrt{\frac{\sin(35 + 23.33) \cdot \sin(35 - 9.46)}{\sin(90 - 23.33) \cdot \sin(90 + 9.46)}} \right]^2}$$

$$K_a = 0.273$$

The retained fill height used in the calculation of the lateral earth pressure and the live load surcharge will be measured to the bottom of the footing regardless of the presence of a shear key.

The wall being designed here does not require a shear key, but the design of a shear key will be shown at the end of the example for informational purposes.

The retained fill height is the combination of the stem height, the additional height added by the 1V:6H back slope over the heel, and the thickness of the footing. The sloped soil starts at the top of the stem, so in our heel calculation, we will only subtract off the 1'-6" stem thickness at the top and not the additional thickness at the bottom due to batter.

$$H_{\text{ret}} = 13 + \frac{8.5 - 2.58 - 1.5}{6} + 1.42 = 15.16 \text{ ft}$$

The stress due to lateral earth pressure is:

$$p_{\text{EH,top}} = \gamma_s \cdot K_a \cdot H_{\text{top}} = 0.120 \cdot 0.273 \cdot 0 = 0 \text{ ksi}$$

$$p_{\text{EH,bot}} = \gamma_s \cdot K_a \cdot H_{\text{ret}} = 0.120 \cdot 0.273 \cdot 15.16 = 0.497 \text{ ksi}$$

The stress varies linearly from the top of the fill to the base of the footing, so the resulting force is:

$$P_{EH} = \frac{1}{2} \cdot p_{EH,bot} \cdot H_{ret} = \frac{1}{2} \cdot 0.497 \cdot 15.16 = 3.77 \text{ k}$$

This force acts on the wall at an angle  $\delta$  from the horizontal based on Coulomb theory. It can be resolved into horizontal and vertical components.

$$P_{EHH} = P_{EH} \cdot \cos(\delta) = 3.77 \cdot \cos(23.33) = 3.46 \text{ k}$$

$$P_{EHV} = P_{EH} \cdot \sin(\delta) = 3.77 \cdot \sin(23.33) = 1.49 \text{ k}$$

The horizontal earth pressure resultant is applied at:

$$y_{EH} = \frac{H_{ret}}{3} = \frac{15.16}{3} = 5.05 \text{ ft above the bottom of footing}$$

See Figure 11.4.2.2 for application of the earth pressure load.

**D. Live Load  
Surcharge (LS)  
[3.11.6.4]**

The horizontal pressure  $p_{LS}$  due to live load surcharge is:

$$p_{LS} = \gamma_{eq} \cdot h_{eq}$$

$\gamma_{eq}$  = equivalent Coulomb fluid pressure

$$\gamma_{eq} = K_a \cdot \gamma_s = 0.273 \cdot 0.120 = 0.033 \frac{\text{kip}}{\text{ft}^3}$$

From AASHTO LRFD Table 3.11.6.4-2, use  $h_{eq} = 2.0$  ft based on a distance from wall backface to edge of traffic  $\geq 1$  ft.

$$p_{LS} = 0.033 \cdot 2 = 0.066 \text{ ksf}$$

Horizontal Component of LS:

$$P_{LSH} = 0.066 \cdot 15.16 \cdot \cos(23.33) = 0.92 \text{ kips}$$

The live load surcharge resultant is applied horizontally at:

$$y_{LS} = \frac{H_{ret}}{2} = \frac{15.16}{2} = 7.58 \text{ ft}$$

Vertical Component of LS applied at back face of EV soil mass:

$$P_{LSV1} = 0.066 \cdot 15.16 \cdot \sin(23.33) = 0.40 \text{ kips}$$

The live load surcharge resultant is applied vertically at the edge of the footing,  $x_{PLSV1} = 8.5$  ft

Vertical component of LS applied to soil mass above heel:

$$p_{LSV2} = 0.120 \cdot 2 \cdot 1 = 0.240 \text{ k/ft}$$

$$P_{LSV2} = 2 \cdot 0.120 \cdot (8.50 - 2.58 - 1.50) = 1.06 \text{ kips}$$

$$x_{PLSV2} = 8.50 - \frac{(8.50 - 2.58 - 1.50)}{2} = 6.29 \text{ ft}$$

See Figure 11.4.2.2 for application of the live load surcharge.

**E. Barrier Collision  
Load (CT)  
[A13.2]**

Per LRFD Article 13.6.2, the barrier collision load is already factored ( $\gamma_{CT}=1.0$ ) and is to be applied only at the Extreme Event II limit state. It will be considered when checking overturning, bearing, sliding, and in design of the footing. A discussion on application of the barrier collision load to the stem design is given in Article 11.4.20. Application of the collision load to the F-barrier reinforcement is shown in Article 13.3.1 of this manual.

For the overturning check, bearing check, sliding check, and footing design, the horizontal vehicle collision force is assumed to be distributed uniformly over the length of one 30.5 foot long panel. The barrier is assumed to be a TL-4 barrier that meets the requirements of NCHRP 350. This requires a design load of 54 kips.

At the bottom of footing:

$$P_{CT} = \frac{54}{30.5} = 1.77 \frac{\text{kip}}{\text{ft}} \text{ of width}$$

$P_{CT}$  is applied at a height  $y_{CT}$  above the footing:

$$y_{CT} = 2.67 + 13.0 + 1.42 = 17.09 \text{ ft}$$

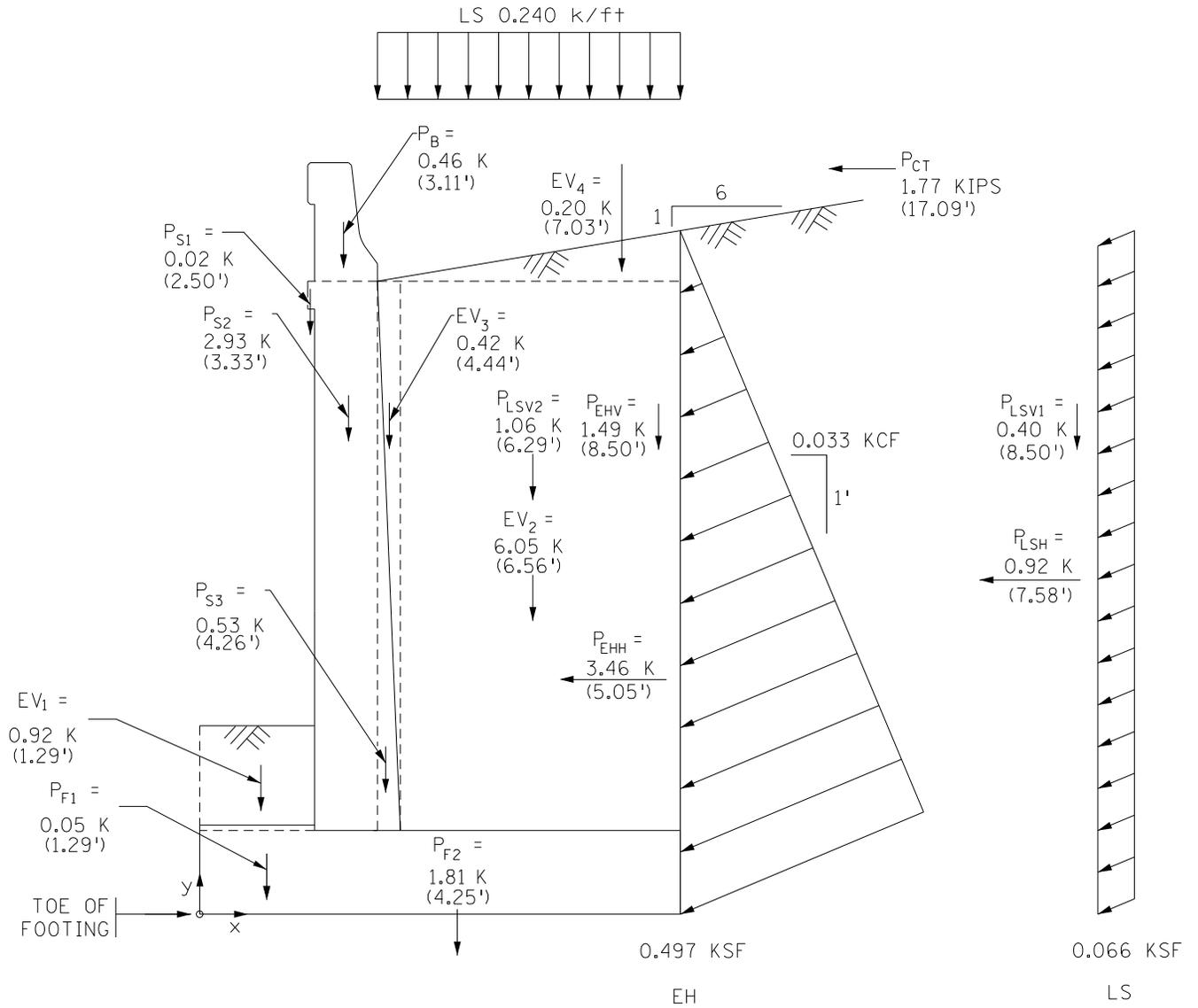


Figure 11.4.2.2

**Table 11.4.2.1 Unfactored Vertical Loads and Moments About Toe of Footing**

Type	Label	Load	Width (ft)	Thickness or height (ft)	Unit Weight (lb/ft <sup>3</sup> )	Load (k/ft)	Lever arm to toe (ft)	Moment (k-ft/ft)
DC	P <sub>B</sub>	Type F Rail	Predetermined linear weight			0.46	3.11	1.43
	P <sub>S1</sub>	Stem coping	0.17	0.67	0.150	0.02	2.50	0.05
	P <sub>S2</sub> *	Stem rectangular part	1.50	13.00	0.150	2.93	3.33	9.76
	P <sub>S3</sub>	Stem tapered part	0.54	13.00	0.150	0.53	4.26	2.26
	P <sub>F2</sub>	Footing	8.50	1.42	0.150	1.81	4.25	7.69
	P <sub>F1</sub> *	Extra thickness on toe	2.58	0.13	0.150	0.05	1.29	0.06
EV	EV <sub>1</sub>	Soil on toe	2.58	2.96	0.120	0.92	1.29	1.19
	EV <sub>2</sub>	Soil on heel rectangular	3.88	13.00	0.120	6.05	6.56	39.69
	EV <sub>3</sub>	Additional soil due to taper	0.54	13.00	0.120	0.42	4.44	1.86
	EV <sub>4</sub>	Extra soil (sloped backfill)	4.42	0.74	0.120	0.20	7.03	1.41
EH	P <sub>EHV</sub>	Vertical active earth pressure	See hand calculations			1.49	8.50	12.67
LS	P <sub>LSV2</sub>	2ft LL surcharge over heel	4.42	2.00	0.120	1.06	6.29	6.67
	P <sub>LSV1</sub>	LL surcharge vertical component	See hand calculations			0.40	8.50	3.40

\*Footing step included in P<sub>F1</sub> for ease of calculations

**Table 11.4.2.2 Unfactored Horizontal Loads and Moments About Bottom of Footing**

Type	Label	Load	Width (ft)	Thickness or height (ft)	Unit Weight (lb/ft <sup>3</sup> )	Load Result (k/ft)	Lever arm to bottom of footing (ft)	Moment (k-ft/ft)
EH	P <sub>EHH</sub>	Horizontal active earth pressure	See hand calculations			3.46	5.05	17.47
LS	P <sub>LSH</sub>	Horizontal load due to LL surcharge	See hand calculations			0.92	7.58	6.97
CT	P <sub>CT</sub>	Barrier (vehicle collision)	See hand calculations			1.77	17.09	30.25

**F. Select Load Modifiers**

For typical retaining walls use:

[1.3.3-1.3.5]

$$\eta_D = 1, \eta_R = 1, \eta_I = 1$$

**G. Select Applicable Load Combinations and Factors**  
[3.4.1]

Table 11.4.2.3 summarizes the load combinations used for design of the wall. Strength Ia and Extreme Event IIa, used to check sliding and overturning, have minimum load factors for the vertical loads and maximum load factors for the horizontal loads. Strength Ib and Extreme Event IIb are used to check bearing and have maximum load factors for both vertical and horizontal loads. Note that live load surcharge (LS) and horizontal earth (EH) are not included in Extreme Event IIa or IIb. The vehicle collision load (CT) is an instantaneous load applied in the same direction as LS and EH. Because of its instantaneous nature, it has the effect of unloading LS and EH. Therefore, the three loads are not additive and only CT is included in the Extreme Event load combinations.

The service limit state is used for the crack control check.

**Table 11.4.2.3 - Load Combinations Considered for Example**

Load Comb.	$\gamma_{DC}$	$\gamma_{EV}$	$\gamma_{LS}$	$\gamma_{EH}$	$\gamma_{CT}$	Application
Strength Ia	0.90	1.00	1.75	1.50	-	Sliding, Overturning
Strength Ib	1.25	1.35	1.75	1.50	-	Bearing, Stem Strength
Extreme IIa	0.90	1.00	-	-	1.00	Sliding, Overturning
Extreme IIb	1.25	1.35	-	-	1.00	Bearing
Service I	1.00	1.00	1.00	1.00	-	Stem Crack Control

### **H. Factor Loads and Moments For Footing Design**

The unfactored loads and moments from Tables 11.4.2.1 and 11.4.2.2 were taken in combination with the load factors in Table 11.4.2.3 to get the factored vertical and horizontal loads to check global stability. An example calculation for the Strength Ia load combination is shown below. Results for other load combinations are shown in Table 11.4.2.4.

As reflected in AASHTO LRFD Figure C11.5.6-3(a), note that the live load surcharge over the heel,  $P_{LSV2}$ , is not used in the Strength Ia or Extreme Event IIa load cases as it would increase the vertical load rather than minimize it. The vertical component  $P_{LSV1}$  and horizontal component  $P_{LSH}$  are always used together.

Also note that the vertical component of the lateral earth pressure,  $P_{EHV}$ , is considered an EH load per AASHTO LRFD Figure C11.5.6-1.

Strength Ia:

$$\begin{aligned}
 P_u &= \gamma_{DCIa} \cdot P_{DC} + \gamma_{EVIa} \cdot P_{EV} + \gamma_{EHIa} \cdot P_{EHV} + \gamma_{LSIa} \cdot P_{LSV} \\
 &= 0.9 \cdot (0.46 + 0.02 + 2.93 + 0.53 + 1.81 + 0.05) \\
 &\quad + 1.0 \cdot (0.92 + 6.05 + 0.42 + 0.20) \\
 &\quad + 1.5 \cdot 1.49 \\
 &\quad + 1.75 \cdot 0.40
 \end{aligned}$$

$$P_u = 15.75 \text{ kips}$$

$$\begin{aligned}
 M_{Pu} &= 0.9 \cdot (1.43 + 0.05 + 9.76 + 2.26 + 7.69 + 0.06) \\
 &\quad + 1.0 \cdot (39.69 + 1.86 + 1.41 + 1.19) \\
 &\quad + 1.5 \cdot 12.67 \\
 &\quad + 1.75 \cdot 3.40
 \end{aligned}$$

$$M_{Pu} = 88.23 \text{ kip-ft}$$

$$H_u = \gamma_{EH1a} \cdot P_{EHH} + \gamma_{LS1a} \cdot P_{LSH}$$

$$H_u = 1.5 \cdot 3.46 + 1.75 \cdot 0.92 = 6.80 \text{ kips}$$

$$M_{Hu} = 1.5 \cdot 17.47 + 1.75 \cdot 6.97 = 38.40 \text{ kip-ft}$$

**Table 11.4.2.4 - Factored Loads and Moments**

Load Combination	Vertical load $P_u$ (kips)	Vertical moment $M_{Pu}$ (k-ft)	Horizontal load $H_u$ (kip)	Horizontal moment $M_{Hu}$ (kip-ft)
Strength Ia	15.75	88.23	6.80	38.40
Strength Ib	22.29	122.79	6.80	38.40
Extreme IIa	12.81	63.28	1.77	30.25
Extreme IIb	17.50	86.17	1.77	30.25
Service	16.34	88.14	4.38	24.44

**I. Check  
Overturning  
(Eccentricity)  
[11.6.3.3]**

The width of footing dimension is designated as “d” in the Bridge Standard Plans for retaining walls. The LRFD Specifications designate the width of the footing as “B”. For this example, the foundation rests on soil.

The current MnDOT CIP Wall Standards were designed using the 2010 AASHTO LRFD code, for which the maximum eccentricity for foundations on soil is B/4. In the 2012 AASHTO LRFD Bridge Design Specifications, the maximum eccentricity for foundations on soil was changed to B/3. This example is based on the current specifications and therefore, the limit of B/3 will be used. For new designs that fall outside the limits of the MnDOT standards, follow the current AASHTO requirements.

**[10.6.3.3]**

$$e_{max} = \frac{B}{3} = \frac{d}{3} = \frac{8.50}{3} = 2.83 \text{ ft}$$

Using the following relationships, compare the actual eccentricity e to  $e_{max}$ :

$$x_r = \frac{M_{Pu} - M_{Hu}}{P_u} \quad \text{Actual } e = \frac{d}{2} - x_r$$

Where  $x_r$  = location of resultant from the toe

For Strength Ia:

$$x_r = \frac{88.23 - 38.40}{15.75} = 3.16 \text{ ft}$$

$$\text{Actual } e = \frac{8.50}{2} - 3.16 = 1.09 \text{ ft} < 2.83 \text{ ft}$$

OK

Results of the check are summarized in Table 11.4.2.5.

**Table 11.4.2.5 Eccentricity Check**

Load Combination	Vertical load $P_u$ (kips)	Vertical moment $M_{Pu}$ (k-ft)	Horizontal moment $M_{Hu}$ (kip-ft)	$x_r$ (ft)	Actual $e$ (ft)	$e_{max}$ (ft)
Strength Ia	15.75	88.23	38.40	3.16	1.09	2.83
Extreme IIa	12.81	63.28	30.25	2.58	1.67	2.83

The footing size is satisfactory for overturning.

**J. Check Bearing**  
[11.6.3.2]

Determine the bearing pressure  $\sigma_v$  at the strength limit state for a foundation on soil.

For Strength Ib:

$$x_r = \frac{M_{Pu} - M_{Hu}}{P_u} = \frac{122.79 - 38.40}{22.29} = 3.79 \text{ ft}$$

$$e = \frac{d}{2} - x_r = \frac{8.50}{2} - 3.79 = 0.46 \text{ ft}$$

$$\text{Effective width } B_{eff} = d - 2e = 8.50 - 2(0.46) = 7.58 \text{ ft}$$

$$\sigma_v = \frac{P_u}{B_{eff}} = \frac{22.29}{7.58} \cdot \left(\frac{1}{2}\right) = 1.47 \text{ tsf}$$

This must be less than the factored bearing resistance provided by the foundations engineer.

Results for the applicable load combinations are shown in Table 11.4.2.6.

**Table 11.4.2.6 Bearing Check**

Load Combination	Vertical load $P_u$ (kips)	Vertical moment $M_{Pu}$ (k-ft)	Horizontal moment $M_{Hu}$ (kip-ft)	$x_r$ (ft)	$e$ (ft)	$B_{eff}$ (ft)	$\sigma_v$ (tsf)
Strength Ib	22.29	122.79	38.40	3.79	0.46	7.58	1.47
Extreme IIb	17.50	86.17	30.25	3.20	1.05	6.40	1.37
Service	16.34	88.14	24.44	3.90	0.35	7.80	1.05

**K. Check Sliding**  
[10.6.3.4]  
[Table 3.11.5.3-1]

The factored horizontal force is checked against the friction resistance between the footing and the soil. If adequate resistance is not provided by the footing, a shear key must be added.

$$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$

From LRFD Table 10.5.5.2.2-1,  $\phi_\tau = 0.80$

$$R_\tau = P_u \tan(\delta) \quad (\text{for cohesionless soils})$$

**[10.6.3.4]**

$$\text{with } \tan(\delta) = \tan(\phi_{\text{found}}) \quad (\text{for CIP footing})$$

For Strength Ia:

$$R_\tau = 15.75 (\tan 32) = 9.84 \text{ kips}$$

$$R_{ep} = 0.0 \quad (\text{No shear key})$$

$$R_R = 0.80 (9.84) + (0.0) = 7.87 \text{ kips} > 6.80 \text{ kips} \quad \text{OK}$$

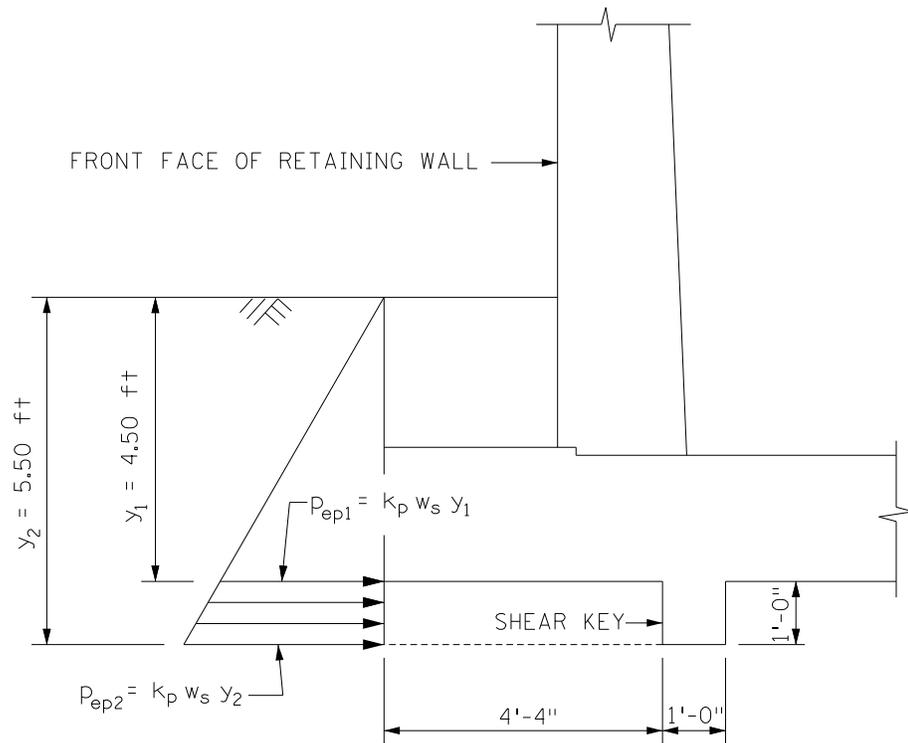
Results for the sliding check are summarized in Table 11.4.2.7

**Table 11.4.2.7 Sliding Check**

Load Combination	Vertical load $P_u$ (kips)	Horizontal load $H_u$ (kips)	$\phi_\tau R_\tau$ (kips)	Check
Strength Ia	15.75	6.80	7.87	OK
Extreme IIa	12.81	1.77	6.40	OK

The footing size is satisfactory for sliding. The design of a shear key will be shown here for informative purposes.

Similar to the Bridge Standard Plans, use a 1'-0" by 1'-0" shear key placed such that the back wall reinforcement will extend into the shear key. Consider only the passive resistance of soil in front of the shear key. Ignore the passive resistance of soil in front of the wall and toe. Refer to Figure 11.4.2.3.



**Figure 11.4.2.3**

As stated earlier, internal friction angle of foundation soil,  $\phi_{\text{found}} = 32^\circ$

Assume a friction angle between the concrete and soil,  $\delta$  as:

$$\delta = \frac{2}{3} \cdot \phi_{\text{found}} = \frac{2}{3} \cdot (32) = 21.33^\circ$$

Toe soil slope  $\beta = 0^\circ$

Angle between face of footing and horizontal  $\theta = 90^\circ$

$$K_p = \frac{\sin^2(\theta - \phi_{\text{found}})}{\sin^2(\theta) \cdot \sin(\theta + \delta) \cdot \left[ 1 - \sqrt{\frac{\sin(\phi_{\text{found}} + \delta) \cdot \sin(\phi_{\text{found}} + \beta)}{\sin(\theta + \delta) \cdot \sin(\theta + \beta)}} \right]^2}$$

$$K_p = \frac{\sin^2(90 - 32)}{\sin^2(90) \cdot \sin(90 + 21.33) \cdot \left[ 1 - \sqrt{\frac{\sin(32 + 21.33) \cdot \sin(32)}{\sin(90 + 21.33) \cdot \sin(90)}} \right]^2}$$

$$K_p = 7.33$$

Then:

$$p_{ep1} = K_p \cdot \gamma_s \cdot y_1 = 7.33 \cdot 0.120 \cdot 4.50 = 3.96 \text{ ksf}$$

$$p_{ep2} = K_p \cdot \gamma_s \cdot y_2 = 7.33 \cdot 0.120 \cdot 5.50 = 4.84 \text{ ksf}$$

$$R_{ep} = \left( \frac{p_{ep1} + p_{ep2}}{2} \right) \cdot (y_2 - y_1) = \left( \frac{3.96 + 4.84}{2} \right) \cdot (5.50 - 4.50) = 4.40 \text{ kips}$$

**[Table 10.5.5.2.2-1]** For the area in front of the shear key, the friction surface is soil on soil, located at the elevation of the bottom of shear key. For this area, use:

$$\phi_{\tau\_sos} = 0.90 \text{ for soil on soil area in front of shear key}$$

The remaining portion of the friction surface is CIP concrete on sand, located at the bottom of shear key and bottom of footing. For this area, use:

$$\phi_{\tau\_cos} = 0.80 \text{ for CIP concrete on sand}$$

For passive resistance from the soil in front of the shear key and below the footing, use:

$$\phi_{ep} = 0.50 \text{ for passive resistance}$$

Determine the weighted average resistance factor,  $\phi_{\tau\_avg}$ , based on footing length.

The shear key is placed to allow the stem back face bars to extend into the key. Then the distance from the front of the toe to the front of the shear key is:

$$x_{sk} = \text{toe length} + \text{stem base} - 3.5"$$

$$x_{sk} = 2.58 + 2.04 - \frac{3.5}{12} = 4.33 \text{ ft}$$

$$\phi_{\tau\_avg} = 0.90 \cdot \left( \frac{4.33}{8.50} \right) + 0.80 \cdot \left( \frac{8.50 - 4.33}{8.50} \right) = 0.85$$

$$R_{\tau} = 9.84 \text{ kips (calculated previously)}$$

For Strength Ia with shear key added:

$$R_R = \phi_{\tau\_avg} \cdot R_{\tau} + \phi_{ep} \cdot R_{ep}$$

$$= 0.85 \cdot 9.84 + 0.50 \cdot 4.40 = 10.56 \text{ kips}$$

**L. Design Footing  
for Shear**  
**[5.13.3.6]**

Design footings to have adequate shear capacity without transverse reinforcement.

**Determine  $d_v$**

As a starting point, assume #6 bars @ 12" ( $A_s = 0.44 \text{ in}^2/\text{ft}$ ) for the top transverse bars in the heel and #5 bars @ 12" ( $A_s = 0.31 \text{ in}^2/\text{ft}$ ) for the bottom transverse bars in the toe. Cover is 3 inches for the top reinforcement and 5 inches for the bottom reinforcement.

Then for the heel:

$$d_{\text{heel}} = 17 - 3 - \frac{0.75}{2} = 13.63 \text{ in}$$

$$a_{\text{heel}} = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.44 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.65 \text{ in}$$

**[5.8.2.9]**  $d_{v\text{heel}} = d_{\text{heel}} - \frac{a}{2} = 13.63 - \frac{0.65}{2} = 13.31 \text{ in}$  GOVERNS

or  $d_{v\text{heel}} = 0.9 \cdot d_e = 0.9 \cdot d_{\text{heel}} = 0.90 \cdot 13.63 = 12.27 \text{ in}$

or  $d_{v\text{heel}} = 0.72 \cdot h = 0.72 \cdot 17.00 = 12.24 \text{ in}$

For the toe:

$$d_{\text{toe}} = 18.5 - 5 - \frac{0.625}{2} = 13.19 \text{ in}$$

$$a_{\text{toe}} = \frac{0.31 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.46 \text{ in}$$

$$d_{v\text{toe}} = 13.19 - \frac{0.46}{2} = 12.96 \text{ in}$$

or  $d_{v\text{toe}} = 0.9 \cdot 13.19 = 11.87 \text{ in}$

or  $d_{v\text{toe}} = 0.72 \cdot 18.50 = 13.32 \text{ in}$  GOVERNS

**Check Heel for Shear**

The vertical loads acting on the heel will be calculated in the same manner as the total vertical load was calculated. The vertical loads acting on the heel will be  $EV_2$ , a revised  $EV_4$  (See Figure 11.4.2.4) that consists of sloped backfill outside of stem/heel juncture only,  $P_{LSV2}$ ,  $P_{EHV}$ , and a revised  $P_{F2}$  which we will call  $P_{F2H}$ , the self-weight of the heel portion of the footing. The loads and moments are summarized in Table 11.4.2.8.

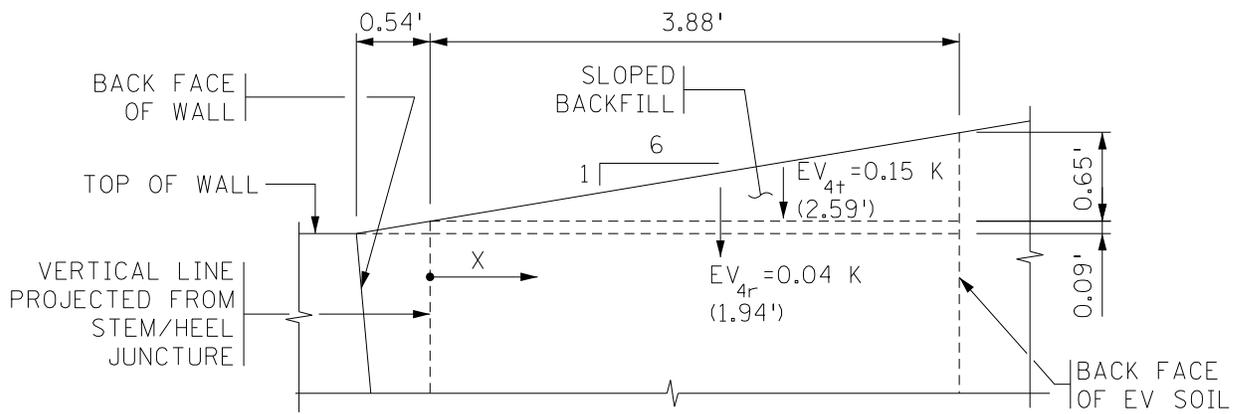


Figure 11.4.2.4

Table 11.4.2.8 Unfactored Vertical Load Components and Moments Acting on Heel

Type	Label	Load	Width (ft)	Thickness or height (ft)	Unit Weight (k/ft <sup>3</sup> )	Load P (k/ft)	Lever arm to stem/heel junction (ft)	Moment (k-ft)
DC	P <sub>F2H</sub>	Footing heel	3.88	1.42	0.150	0.83	1.94	1.61
EV	EV <sub>2</sub>	Soil on heel rectangular	3.88	13.00	0.120	6.05	1.94	11.74
	EV <sub>4r</sub>	Sloped backfill rectangle	3.88	0.09	0.120	0.04	1.94	0.08
	EV <sub>4t</sub>	Sloped backfill triangle	3.88	0.65	0.120	0.15	2.59	0.39
EH	P <sub>EHV</sub>	Vertical active earth pressure	See hand calculations			1.49	3.88	5.78
LS	P <sub>LSV2</sub>	2ft LL surcharge	3.88	2.00	0.120	0.93	1.94	1.80
	P <sub>LSV1</sub>	LL surcharge vertical component	See hand calculations			0.40	3.88	1.55

These loads need to be factored and then the upward vertical force from the trapezoidal bearing pressure acting on the heel can be calculated. The largest net vertical force will be used to design the heel.

An example calculation is shown here for Strength Ia:

$$\begin{aligned}
 P_{\text{uheel}} &= \gamma_{DC} \cdot P_{F2H} + \gamma_{EV} \cdot (EV_2 + EV_{4r} + EV_{4t}) + \gamma_{EH} \cdot P_{EHV} + \gamma_{LS} \cdot P_{LSV1} \\
 &= 0.90 \cdot 0.83 + 1.0 \cdot (6.05 + 0.04 + 0.15) + 1.5 \cdot 1.49 + 1.75 \cdot 0.40 \\
 &= 9.92 \text{ kips}
 \end{aligned}$$

We need to calculate the upward bearing pressure acting on the heel that can be subtracted off of the downward vertical loads to get a net vertical load.

**[10.6.5]**

Although the wall will be supported on soil, the trapezoidal bearing stress distribution is used in the structural design of the footing. This will produce larger upward forces acting on the toe and smaller upward forces acting on the heel, both of which are conservative.

Calculate the maximum and minimum vertical pressure for the Strength Ia load case (See Figure 11.4.2.5). The following equation is used when the resultant is within the middle one-third of the base.

**[11.6.3.2]**

$$\sigma_v = \frac{\Sigma P}{B} \left( 1 \pm 6 \cdot \frac{e}{B} \right)$$

$$\Sigma P = 15.75 \text{ kips (from Table 11.4.2.4)}$$

$$e = 1.09 \text{ ft (from Table 11.4.2.5)}$$

$$B = 8.5 \text{ ft (width of the footing)}$$

$$\sigma_{pmax} = \frac{15.75}{8.5} \left( 1 + 6 \cdot \frac{1.09}{8.5} \right) = 3.28 \frac{\text{kip}}{\text{ft}^2}$$

$$\sigma_{pmin} = \frac{15.75}{8.5} \left( 1 - 6 \cdot \frac{1.09}{8.5} \right) = 0.43 \frac{\text{kip}}{\text{ft}^2}$$

Next, use linear interpolation to calculate the vertical pressure at the heel/back face of stem junction.

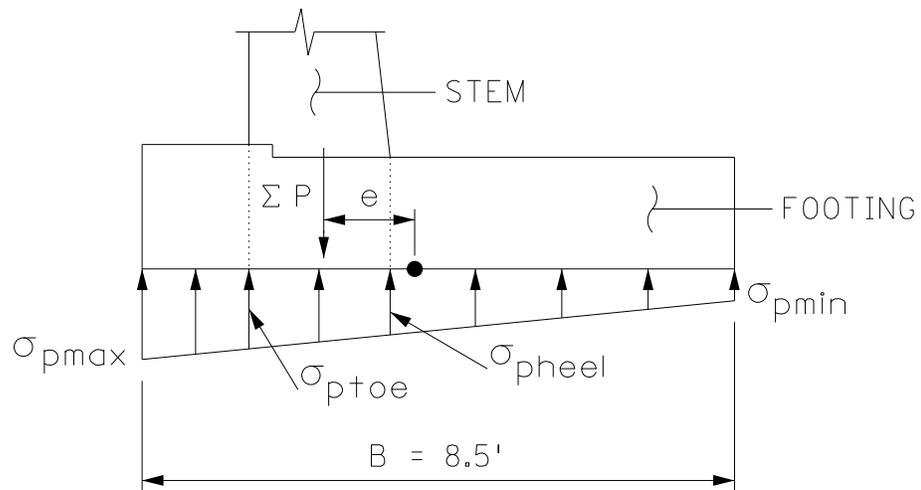
$$\sigma_{pheel} = 0.43 + (3.28 - 0.43) \cdot \frac{3.88}{8.5} = 1.73 \text{ ksf}$$

Since the upward pressure varies linearly over the heel between  $\sigma_{pmin}$  and  $\sigma_{pheel}$ , we can average these two pressures and use that value to calculate the upward vertical force on the heel.

$$P_{up-heel} = \frac{0.43 + 1.73}{2} \cdot 3.88 = 4.19 \text{ kips}$$

So the net vertical load acting on the heel is:

$$P_{unetH} = P_{uheel} - P_{up-heel} = 9.92 - 4.19 = 5.73 \text{ kips}$$



**Trapezoidal Pressure for Footing Structural Design**  
**Figure 11.4.2.5**

Use the following for instances where the resultant is outside the middle one-third of the base, to account for when the bearing stress is triangular and the minimum heel pressure is zero.

[11.6.3.2]

$$\sigma_{vmax} = \frac{2 \cdot \Sigma P}{3 \cdot \left(\frac{B}{2} - e\right)}$$

$$\sigma_{vmin} = 0 \text{ ksf}$$

The vertical loads are summarized in Table 11.4.2.9.

**Table 11.4.2.9 Factored Vertical Loading on Heel**

Load Combination	Vertical load $P_{uheel}$ (kips)	Max. vertical pressure (ksf)	Min. vertical pressure (ksf)	Distance $x_0$ from toe to 0 pressure (ft)	Heel pressure at jct. of stem/heel (ksf)	Upward vertical load on heel (kip)	Net vertical load $P_{unet}$ (kip)
Strength Ia	9.92	3.28	0.43	na	1.73	4.19	5.73
Strength Ib	14.02	3.47	1.77	na	2.55	8.38	5.64
Extreme IIa	6.99	3.31	0.00	7.74	1.33	2.07	4.92
Extreme IIb	9.46	3.58	0.53	na	1.92	4.75	4.71
Service	9.89	2.40	1.45	na	1.88	6.46	3.43

Since the heel length of 3.88 ft is more than  $3d_v = 3.28$  ft, we cannot use the simplifications from 5.8.3.4.1 and must calculate  $\beta$ .

Therefore, we will need to calculate the downward moment caused by these loads in order to calculate  $\epsilon_s$  below. The moment will also be used to size the flexural reinforcement in Article 11.4.2M.

$$\begin{aligned}
 M_{u\text{heel}} &= \gamma_{DC} \cdot M_{DC} + \gamma_{EV} \cdot M_{EV} + \gamma_{EH} \cdot M_{EH} + \gamma_{LS} \cdot M_{LS} \\
 &= 0.90 \cdot 1.61 + 1.0 \cdot (11.74 + 0.08 + 0.39) + 1.5 \cdot 5.78 + 1.75 \cdot 1.55 \\
 &= 25.04 \text{ kip - ft}
 \end{aligned}$$

We also need to calculate the moment caused by the upward pressure.

$$\begin{aligned}
 M_{\text{up-heel}} &= 0.43 \cdot 3.88 \cdot \frac{3.88}{2} + \frac{1}{2} \cdot (1.73 - 0.43) \cdot 3.88 \cdot \frac{3.88}{3} \\
 &= 6.50 \text{ kip - ft}
 \end{aligned}$$

$$M_{\text{unet}} = M_u - M_{\text{up-heel}} = 25.04 - 6.50 = 18.54 \text{ kip - ft}$$

The vertical moments are summarized in Table 11.4.2.10.

**Table 11.4.2.10 Vertical Moments Acting on Heel**

Load Combination	Downward vertical moment $M_{u\text{heel}}$ (k-ft)	Upward moment from bearing pressure $M_{\text{up-heel}}$ (kip-ft)	Net vertical moment $M_{\text{unet}}$ (kip-ft)
Strength Ia	25.04	6.50	18.54
Strength Ib	33.03	15.28	17.75
Extreme IIa	13.66	3.34	10.32
Extreme IIb	18.50	7.48	11.02
Service	22.95	11.99	10.96

**[5.8.3.3]**

We can then calculate the shear capacity of the heel, assuming no transverse reinforcement.

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

We will design the footing so that shear reinforcement is not needed, so we will use equation 5.8.3.4.2-2 to calculate  $\beta$ . Axial compression will also be ignored.

$$\beta = \frac{4.8}{(1 + 750\epsilon_s)} \cdot \frac{51}{(39 + s_{xe})}$$

$$\epsilon_s = \frac{\frac{M_{u\text{heel}}}{d_v} + V_{u\text{heel}}}{E_s A_s}$$

**[C5.13.3.6.1]**

The critical section for shear on the heel is at the heel/back face of stem junction.

$$M_{\text{unet}} = 18.54 \text{ kip-ft}$$

$$V_{\text{uheel}} = P_{\text{unet}} = 5.73 \text{ kips}$$

$$d_v = 13.31 \text{ in from before}$$

$$A_s = 0.44 \text{ in}^2 \text{ assuming \#6 bars @ 12" for top transverse reinforcement}$$

$$\epsilon_s = \frac{\frac{18.54 \cdot 12}{13.31} + 5.73}{29000 \cdot 0.44} = 0.00176$$

Next, determine  $s_{xe}$ :

$$s_{xe} = s_x \cdot \frac{1.38}{a_g + 0.63}$$

Referring to AASHTO Figure 5.8.3.4.2-3(a),

$$s_x = d_v = 13.31 \text{ in}$$

$a_g$  = maximum aggregate size = 0.75 in (smallest max aggregate size for Concrete Mix 1G52, which is conservative)

$$s_{xe} = 13.31 \cdot \frac{1.38}{0.75 + 0.63} = 13.31 \text{ in}$$

$$\beta = \frac{4.8}{1 + 750 \cdot 0.00176} \cdot \frac{51}{39 + 13.31} = 2.02$$

Calculate shear capacity of a one foot wide strip of the footing:

**[5.5.4.2.1]**

$$V_c = 0.0316 \cdot 2.02 \cdot \sqrt{4} \cdot 12 \cdot 13.31 = 20.39 \text{ kips}$$

$$\phi = 0.90 \text{ for shear on normal weight concrete}$$

$$\phi V_c = 0.90 \cdot 20.39 = 18.35 \text{ kips}$$

Compare the shear capacity to the factored shear demand:

$$\phi V_c = 18.35 \text{ kips} > V_{\text{uheel}} = 5.73 \text{ kips} \quad \text{OK}$$

This procedure can be repeated for all load cases. The results are summarized in Table 11.4.2.11.

Table 11.4.2.11 Heel Shear Check

Load Combination	$V_{u\text{heel}}$ (kips)	Net vertical moment $M_{u\text{net}}$ (kip-ft)	$\epsilon_s$	$\beta$	$\phi V_c$ (kip/ft)	$\phi V_c > V_{u\text{heel}}?$
Strength Ia	5.73	18.54	0.00176	2.02	18.35	YES
Strength Ib	5.64	17.75	0.00170	2.06	18.71	YES
Extreme IIa	4.92	10.32	0.00111	2.55	23.17	YES
Extreme IIb	4.71	11.02	0.00115	2.51	22.81	YES
Service	3.43	10.96	0.00104	2.63	23.90	YES

**Check Toe for Shear**

The shear demand on the toe will be calculated by using the upward bearing pressure and ignoring the downward load from the soil cover over the toe. The maximum and minimum bearing pressures across the width of the footing have already been calculated. We will need to calculate the bearing pressure at  $d_v$  from the front face of the stem.

$$d_{v\text{toe}} = 13.32 \text{ in} = 1.11 \text{ ft}$$

$$\sigma_{p\text{max}} = 3.28 \text{ kip/ft}^2$$

$$\sigma_{p\text{min}} = 0.43 \text{ kip/ft}^2$$

$$\sigma_{p\text{dvt}\text{toe}} = 3.28 - (3.28 - 0.43) \cdot \left( \frac{2.58 - 1.11}{8.5} \right) = 2.79 \text{ ksf}$$

$$P_{\text{udvt}\text{toe}} = \frac{2.79 + 3.28}{2} \cdot (2.58 - 1.11) = 4.46 \text{ kips}$$

$$V_{\text{udvt}\text{toe}} = P_{\text{udvt}\text{toe}} = 4.46 \text{ kips}$$

The shear capacity of the toe will be calculated in the same manner as it was for the heel:

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v d_v$$

Since the toe length of 2.58 ft is less than  $3d_v = 3.33$  ft, the provisions of 5.8.3.4.1 can be used and  $\beta$  can be taken as 2.

Calculate shear capacity of a one foot wide strip of the footing:

$$V_c = 0.0316 \cdot 2.0 \cdot \sqrt{4} \cdot 12 \cdot 13.32 = 20.2 \text{ kips}$$

$$\phi = 0.90 \text{ for shear on normal weight concrete}$$

$$\phi V_c = 0.90 \cdot 20.2 = 18.2 \text{ kips}$$

Compare the shear capacity to the factored shear demand.

$$\phi V_c = 18.2 \text{ kips} > V_{\text{utoe}} = 4.46 \text{ kips}$$

OK

Since the simplification of  $\beta=2$  is in use, the shear capacity of the section will be the same for all load combinations. The bearing pressure will depend on the load combinations.

Table 11.4.2.12 summarizes the results of the toe shear check.

**Table 11.4.2.12 Toe Shear Check**

Load Combination	Max. vertical pressure (ksf)	Min. vertical pressure (ksf)	Distance $x_0$ from heel to 0 pressure (ft)	Pressure at front face of stem (ksf)	Pressure at $d_v$ from front face of stem (ksf)	$V_{\text{utoe}}$ (kips)	$\phi V_c$ (kips)	$\phi V_c \geq V_{\text{utoe}}$
Strength Ia	3.28	0.43	na	2.41	2.79	4.46	18.2	YES
Strength Ib	3.47	1.77	na	2.95	3.18	4.89	18.2	YES
Extreme IIa	3.31	0.00	7.74	2.21	3.03	4.66	18.2	YES
Extreme IIb	3.58	0.53	na	2.65	3.05	4.87	18.2	YES
Service	2.40	1.45	na	2.11	2.24	3.41	18.2	YES

**M. Design Footing Reinforcement**  
[5.13.3.4]

Each mat of reinforcement is checked to ensure that it has adequate capacity and that minimum reinforcement checks are satisfied.

The top transverse reinforcement is designed by assuming that the heel acts as a cantilever loaded by the vertical loads above the heel. The upward bearing pressure is subtracted off the downward vertical loads.

The bottom transverse reinforcement is designed by assuming that the toe acts as a cantilever loaded by the upward bearing pressure on the heel. The soil cover above the toe is ignored.

The critical section for flexure in the footing is on either side of the stem.

**Top Transverse Reinforcement**

Assuming  $\phi = 0.90$  and using  $M_{\text{unet}}$  calculated for the Strength Ia check, set up the flexural capacity equation to solve for required steel area:

$$M_u = \phi M_n = \phi \cdot A_s \cdot f_y \left( d_s - \frac{a}{2} \right)$$

$$M_u = \phi \cdot A_s \cdot f_y \left( d_s - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b} \right)$$

$$M_u = 0.90 \cdot A_s \cdot 60 \cdot \left( d_s - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 12} \right) \cdot \left( \frac{1}{12} \right)$$

$$3.309 \cdot A_s^2 - 4.5 \cdot d_s \cdot A_s + M_u = 0$$

From the shear check of the heel,  $M_{\text{unet}} = 18.54$  k-ft

For 3" clear cover and #7 bars,  $d_s = 13.63$  in

Substituting and solving for  $A_s$ , we get:

$$\text{Required } A_s = 0.31 \text{ in}^2/\text{ft}$$

$$\text{Try #6 bars @ 12"}, A_s = 0.44 \text{ in}^2/\text{ft}$$

Check that assumed  $\phi = 0.90$  is correct:

$$c = \frac{A_s \cdot f_y}{0.85f'_c \cdot \beta_1 \cdot b} = \frac{0.44 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 0.76 \text{ in}$$

### [5.7.2.1]

Concrete compression strain limit  $\epsilon_c = 0.003$

### [Table C5.7.2.1-1]

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (13.63 - 0.76) \left( \frac{0.003}{0.76} \right) = 0.0508 > \epsilon_{tl} = 0.005$$

Therefore,  $\phi = 0.90$

### Check Minimum Reinforcement

### [5.7.3.3.2]

Determine the cracking moment:

$$\lambda = 1.0 \text{ for normal weight concrete}$$

### [5.4.2.6]

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{4} = 0.48 \text{ ksi}$$

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (17)^3 = 4913 \text{ in}^4$$

$$y_t = \frac{1}{2} \cdot t = \frac{1}{2} \cdot 17 = 8.5 \text{ in}$$

Using  $\gamma_1 = 1.6$  and  $\gamma_3 = 0.67$  for ASTM 615 Grade 60 reinforcement,

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot \frac{f_r \cdot I_g}{y_t} = 0.67 \cdot 1.6 \cdot \frac{0.48 \cdot 4913}{8.5 \cdot (12)} = 24.8 \text{ kip-ft}$$

The capacity of the section must be greater than or equal to the smaller of:

$$M_{cr} = 24.8 \text{ kip-ft}$$

$$\text{or } 1.33 \cdot M_u = 1.33 \cdot 18.54 = 24.7 \text{ kip-ft}$$

GOVERNS

The capacity of the top mat of reinforcement is:

$$M_r = \phi A_s f_y (d_s - a/2)$$

For #6 bars,  $d_s = 13.63$  in

$$M_r = 0.9 \cdot (0.44) \cdot (60) \cdot \left[ 13.63 - \frac{0.44 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)} \right] \cdot \frac{1}{12}$$

$$M_r = 26.3 \text{ kip-ft} > 24.7 \text{ kip-ft} \quad \text{OK}$$

Use #6 bars @ 12" ( $A_s = 0.44 \text{ in}^2/\text{ft}$ ) for top transverse reinforcement in the footing.

### Bottom Transverse Reinforcement

The moment acting on the toe from the upward bearing pressure can be determined based on the data that was calculated for the Strength Ib toe shear check.

$$M_u = V_u \cdot \text{moment arm}$$

For a toe length of 2.58 ft and knowing the vertical pressures at either end of the toe, the moment can be calculated as:

$$M_u = 2.95 \cdot 2.58 \cdot \frac{2.58}{2} + \frac{1}{2} (3.47 - 2.95) \cdot 2.58 \cdot \frac{2}{3} \cdot 2.58 = 10.97 \text{ kip-ft}$$

For 5" clear cover and #5 bars,  $d_s = 13.19$  in

$$\text{Again use: } 3.309 \cdot A_s^2 - 4.5 \cdot d_s \cdot A_s + M_u = 0$$

Substituting and solving for  $A_s$ , we get:

$$\text{Required } A_s = 0.19 \text{ in}^2/\text{ft}$$

$$\text{Try #5 bars @12", } A_s = 0.31 \text{ in}^2/\text{ft}$$

### Check Minimum Reinforcement

[5.7.3.3.2]

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (18.50)^3 = 6332 \text{ in}^4$$

$$y_t = \frac{1}{2} \cdot t = \frac{1}{2} \cdot 18.50 = 9.25 \text{ in}$$

Use  $\gamma_1 = 1.6$  and  $\gamma_3 = 0.67$  for ASTM 615 Grade 60 reinforcement.

Then:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot \frac{f_r \cdot I_g}{y_t} = 0.67 \cdot 1.6 \cdot \frac{0.48 \cdot 6332}{9.25 \cdot (12)} = 29.4 \text{ kip-ft}$$

The capacity of the section must be greater than or equal to the smaller of:

$$M_{cr} = 29.4 \text{ kip-ft}$$

$$\text{or } 1.33 \cdot M_u = 1.33 \cdot 10.97 = 14.6 \text{ kip-ft} \quad \underline{\text{GOVERNS}}$$

Compute the capacity of the provided steel:

$$M_r = \phi A_s f_y (d_s - a/2)$$

$$M_r = 0.9 (0.31) \cdot (60) \cdot \left[ 13.19 - \frac{0.31 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)} \right] \cdot \left( \frac{1}{12} \right)$$

$$M_r = 18.1 \text{ ft-kips} > 14.6 \text{ kip-ft} \quad \underline{\text{OK}}$$

Use #5 bars @ 12" ( $A_s = 0.31 \text{ in}^2/\text{ft}$ ) for bottom transverse reinforcement in the footing.

### Longitudinal Reinforcement

[5.10.8]

Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirements.

$$A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.30 \cdot 102 \cdot 18.5}{2 \cdot (102 + 18.5) \cdot 60} = 0.17 \text{ in}^2$$

$$0.11 \leq A_s \leq 0.60$$

Use #5 bars @ 12" ( $A_s = 0.31 \text{ in}^2/\text{ft}$ ) for top and bottom longitudinal reinforcement in the footing.

### N. Determine Loads For Wall Stem Design

The loads on the stem at the top of the footing can now be determined to arrive at the design forces for the wall. The stem will be designed for at-rest earth pressure. This will govern the design of the back face vertical bars.

We will calculate the at-rest earth pressure coefficient in accordance with the geotechnical design assumptions given on the MnDOT Standard Plan Sheet 5-297.639, which contains the cast-in-place retaining wall basis for design:

$$K_O = 1 - \sin(\phi_{\text{fret}})$$

But the coefficient must be modified to account for the sloped backfill:

$$K_{O\beta} = K_O \cdot (1 + \sin(\beta))$$

This modification is based on an equation used in the Danish Code (Danish Geotechnical Institute 1978).

$$K_O = 1 - \sin(35) = 0.426$$

$$K_{O\beta} = 0.426 \cdot (1 + \sin(9.46)) = 0.496$$

The loading at any height along the stem, where the height  $y_{stem}$  is measured below the groundline, will be due to the combination of lateral earth pressure and live load surcharge.

Lateral Earth Pressure:

$$V_{EH} = \frac{1}{2} \cdot \gamma_s \cdot K_{O\beta} \cdot y_{stem}^2$$

$$\begin{aligned} M_{EH} &= V_{EH} \cdot \frac{y_{stem}}{3} \\ &= \frac{1}{6} \cdot \gamma_s \cdot K_{O\beta} \cdot y_{stem}^3 \end{aligned}$$

Although this force acts at an angle parallel to the backfill slope from the horizontal, we will conservatively apply it horizontally for stem design.

The horizontal load due to the live load surcharge can be computed similarly:

$$V_{LS} = \gamma_s \cdot K_O \cdot 2 \cdot y_{stem}$$

$$\begin{aligned} M_{LS} &= V_{LS} \cdot \frac{y_{stem}}{2} \\ &= \gamma_s \cdot K_O \cdot 2 \cdot \frac{y_{stem}^2}{2} \end{aligned}$$

### ***O. Determine Load Combinations For Stem Design***

By inspection, we can see that Strength Ia and Extreme Event IIa are the possible load combinations that could govern the design of the stem since they maximize the horizontal loads.

In checking global stability and footing design, it was assumed that the 54 kip CT load was distributed uniformly over a 30.5 foot panel. For stem design, this assumption is not appropriate. The collision load will be distributed over some length less than 30.5 feet. In order to properly consider the collision load, the stem was analyzed using a finite element

model and found to be sufficient. Consequently, the Extreme Event load cases will not be considered in the design of the stem.

**P. Determine Factored Loads and Moments For Stem Design**

The load factors from Table 11.4.2.3 are to be used with the following modification:

- Apply a factor of 1.35 to horizontal earth pressure in the at-rest condition per AASHTO LRFD Table 3.4.1-2.

Factoring the loads for the Strength Ia load combination,

$$\begin{aligned} V_u &= \gamma_{EH\text{Ia\_AR}} \cdot V_{EH} + \gamma_{LS\text{Ia}} \cdot V_{LS} \\ &= 1.35 \cdot V_{EH} + 1.75 \cdot V_{LS} \end{aligned}$$

Factoring the moments for the Strength Ia load combination:

$$\begin{aligned} M_u &= \gamma_{EH\text{Ia\_AR}} \cdot M_{EH} + \gamma_{LS\text{Ia}} \cdot M_{LS} \\ &= 1.35 \cdot M_{EH} + 1.75 \cdot M_{LS} \end{aligned}$$

We then need to determine the design moment at each wall height based on the minimum reinforcement provisions.

**[5.7.3.3.2]**

Compute the cracking moment:

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_c$$

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.241 \cdot 1.0 \cdot \sqrt{4} = 0.48 \text{ ksi}$$

$$S_c = \frac{bh^2}{6}$$

b = 1 ft wide section of wall we are designing

h = thickness of stem at height considered

$$= 18 + \frac{y_{\text{stem}} \cdot 12}{24} \quad \text{where } y_{\text{stem}} \text{ is in feet}$$

$\gamma_3 = 0.67$  for ASTM A615 Grade 60 reinforcement

$\gamma_1 = 1.6$  for all other concrete structures

$$\begin{aligned} M_{cr} &= \left[ 0.67 \cdot 1.6 \cdot 0.48 \cdot \frac{\left( 12 \cdot \left( 18 + \frac{y_{\text{stem}}}{2} \right)^2 \right)}{6} \right] \cdot \frac{1}{12} \\ &= 27.8 + 1.54 \cdot y_{\text{stem}} + 0.02144 \cdot y_{\text{stem}}^2 \end{aligned}$$

where  $y_{\text{stem}}$  is in feet and  $M_{cr}$  is in kip-ft

Then the design moment,  $M_{des}$ , will be:

If  $M_u \geq M_{cr}$ ,  $M_{des} = M_u$

If  $M_u < M_{cr}$  and  $1.33M_u \leq M_{cr}$ ,  $M_{des} = 1.33M_u$

If  $M_u < M_{cr}$  and  $1.33M_u > M_{cr}$ ,  $M_{des} = M_{cr}$

Tables 11.4.2.13 and 11.4.2.14 summarize the stem moments and shears.

**Table 11.4.2.13 Design Moment on Stem**

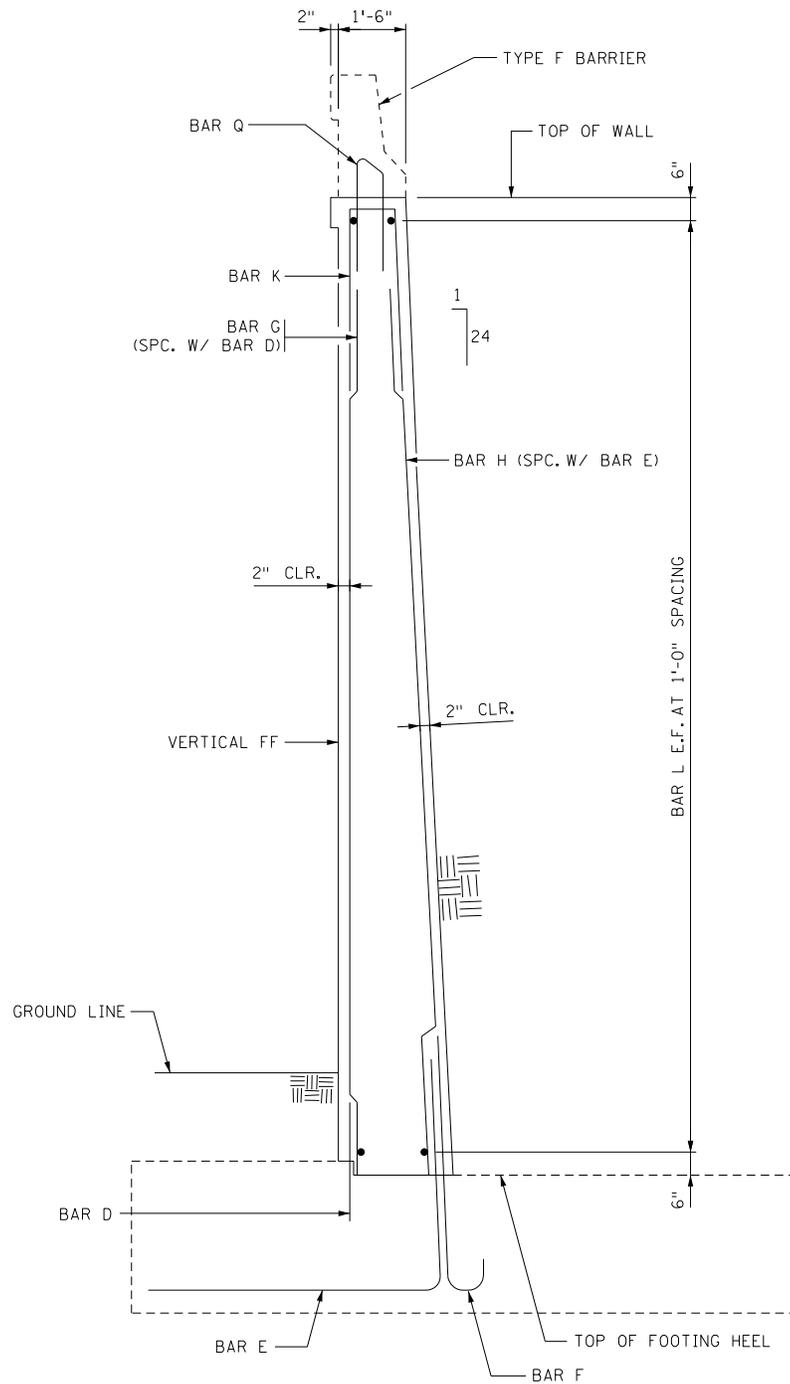
$y_{stem}$ (ft)	$M_{EH}$ (kip-ft)	$M_{LS}$ (kip-ft)	$M_{service}$ (kip-ft)	$M_u$ (kip-ft)	$1.33 \cdot M_u$ (kip-ft)	$M_{cr}$ (kip-ft)	$M_{des}$ (kip-ft)
2	0.1	0.2	0.3	0.5	0.7	31.0	0.7
4	0.6	0.8	1.4	2.2	2.9	34.3	2.9
6	2.1	1.8	3.9	6.0	8.0	37.8	8.0
8	5.1	3.3	8.4	12.7	16.9	41.5	16.9
9	7.2	4.1	11.3	16.9	22.5	43.4	22.5
10	9.9	5.1	15.0	22.3	29.7	45.3	29.7
11	13.2	6.2	19.4	28.7	38.2	47.3	38.2
12	17.1	7.4	24.5	36.0	47.9	49.4	47.9
13	21.8	8.6	30.4	44.5	59.2	51.4	51.4

**Table 11.4.2.14 Design Shear on Stem**

$y_{stem}$ (ft)	$V_{EH}$ (kip)	$V_{LS}$ (kip)	$V_{service}$ (kip)	$V_u$ (kip)
2	0.1	0.2	0.3	0.5
4	0.5	0.4	0.9	1.4
6	1.1	0.6	1.7	2.5
8	1.9	0.8	2.7	4.0
9	2.4	0.9	3.3	4.8
10	3.0	1.0	4.0	5.8
11	3.6	1.1	4.7	6.8
12	4.3	1.2	5.5	7.9
13	5.0	1.3	6.3	9.0

**P. Wall Stem  
Design –  
Investigate  
Strength Limit  
State**

Since this example is based on the current MnDOT LRFD Cast-In-Place Retaining Wall Standards (Standard Plan Sheets 5-297.620 to .635), the bar designation from the standards will be used. Each bar is designated by a letter of the alphabet. See Figure 11.4.2.6.



**Figure 11.4.2.6**

In determination of the back face reinforcement, the bars to be considered are bars E, F, H, and K. Bars E, H, and K are lapped together with Class B lap splices and are spaced at 12 inches. Bar F is also spaced at 12 inches, alternating with Bar E.

First, we will determine the reinforcement required at the base of the stem. Then for Bar F, we will calculate the location above the stem/footing interface where it can be dropped.

Assuming  $\phi = 0.90$ , set up the flexural capacity equation to solve for required steel area:

$$M_u = \phi M_n = \phi \cdot A_s \cdot f_y \left( d_s - \frac{a}{2} \right)$$

$$M_u = \phi \cdot A_s \cdot f_y \left( d_s - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b} \right)$$

$$M_u = 0.90 \cdot A_s \cdot 60 \cdot \left( d_s - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 12} \right) \cdot \left( \frac{1}{12} \right)$$

$$3.309 \cdot A_s^2 - 4.5 \cdot d_s \cdot A_s + M_u = 0$$

Assuming #5 bars with 2" clear cover at the stem/footing interface,  
 $d_s = 24.5 - 2 - 0.5 \cdot (0.625) = 22.19$  in  
 $M_{des} = 51.4$  k-ft

Substituting and solving for  $A_s$ , we get:

$$\text{Required } A_s = 0.52 \text{ in}^2/\text{ft}$$

Try #5 bars @ 6",  $A_s = 0.62 \text{ in}^2/\text{ft}$ ,  $\phi M_n = 60.6$  k-ft

This accounts for Bar E spaced at 12" and Bar F spaced at 12".

Check that assumed  $\phi = 0.90$  is correct:

$$c = \frac{A_s \cdot f_y}{0.85 f'_c \cdot \beta_1 \cdot b} = \frac{0.62 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 1.07 \text{ in}$$

### [5.7.2.1]

Concrete compression strain limit  $\epsilon_c = 0.003$

### [Table C5.7.2.1-1]

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (22.19 - 1.07) \left( \frac{0.003}{1.07} \right) = 0.059 > \epsilon_{tl} = 0.005$$

Therefore,  $\phi = 0.90$

An initial estimate for the point at which Bar F is no longer needed is made using Table 11.4.2.13. Since dropping Bar F will leave us with half of the reinforcement provided at the base of the stem, scan the table for the height at which  $M_{des}$  is approximately half the value of  $\phi M_n$  at the base of the stem. The table shows that this occurs at  $y_{stem} = 10'$ , where

$M_{des} = 29.7$  kip-ft. Therefore, the point above the footing where Bar F is no longer needed is around:

$$y_{stem} = 10 \text{ ft, at which } d_s = 20.69 \text{ in}$$

Solving for  $A_s$ , we get:

$$\text{Required } A_s = 0.32 \text{ in}^2/\text{ft}$$

For #5 bars at 12",  $A_s = 0.31 \text{ in}^2/\text{ft}$ . Therefore, Bar F is still needed at 10 ft below the ground line. Check if Bar F is needed at  $y_{stem} = 9.75$  ft.

$$\text{At } y_{stem} = 9.75 \text{ ft, the depth } d_s = 20.57 \text{ in, and } M_{des} = 27.9 \text{ k-ft.}$$

Solving for  $A_s$ , we get:

$$\text{Required } A_s = 0.31 \text{ in}^2/\text{ft} \quad \underline{\text{OK}}$$

### [5.11.1.2.1]

Reinforcement is required to extend beyond the point at which it is no longer required to the greater of:

$$d_s = 20.57 \text{ in} \quad \underline{\text{GOVERNS}}$$

$$\text{or } 15 \cdot d_b = 15 \cdot 0.625 = 9.38 \text{ in}$$

Use an extension of 21 in = 1.75 ft

Therefore, provide a projection  $x_{proj}$  above the top of footing for Bar F of:

$$x_{proj} = 13 - 9.75 + 1.75 = 5.00 \text{ ft}$$

Bar F must be fully developed at the stem/footing interface. For epoxy coated #5 bars the basic development length  $\ell_{db}$  is:

$$\ell_{db} = \frac{2.4 \cdot d_b \cdot f_y}{\sqrt{f'_c}} = \frac{2.4 \cdot 0.625 \cdot 60}{\sqrt{4.0}} = 45.0 \text{ in.}$$

The modification factors to the development length are:

$\lambda_{cf} = 1.2$  for epoxy coated bars with cover greater than 3 bar diameters and clear spacing greater than 6 bar diameters

$\lambda_{rl} = 1.0$  for vertical bars

$\lambda = 1.0$  for normal weight concrete

$\lambda_{er} = 1.0$  taken conservatively assuming  $A_{sprovided} = A_{srequired}$

For determination of  $\lambda_{rc}$ :

$c_b = 2.31$  in. (governed by 2.0 clear + 0.5 · bar diameter)

$A_{tr} = 0$  since there are no bars that cross the potential splitting planes

Then  $k_{tr} = 0$

$$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}} = \frac{0.625}{2.31 + 0} = 0.27 < 0.4$$

So  $\lambda_{rc} = 0.4$

Then the development length  $\ell_d$  is:

$$\ell_d = \frac{\ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = \frac{45.0 \cdot (1.0 \cdot 1.2 \cdot 0.4 \cdot 1.0)}{1.0} = 21.60 \text{ in.}$$

Therefore, projecting Bar F into the stem 5 feet ( $y_{stem} = 8 \text{ ft}$ ) will provide adequate back face reinforcement in the stem.

Table 11.4.2.15 summarizes the Strength Limit state check for the stem back face reinforcement over the stem height starting at  $y_{stem} = 8 \text{ ft}$ .

**Table 11.4.2.15 Moment Capacity of Stem**

$y_{stem}$ (ft)	Wall thickness (in)	$d_s$ (in)	$M_{des}$ (kip-ft)	Reqd $A_s$ (in <sup>2</sup> /ft)	Actual $A_s$ (in <sup>2</sup> /ft)	$\phi M_n$ (kip-ft)	C/D
8	22.00	19.69	16.9	0.19	0.31	27.1	1.60
9	22.50	20.19	22.5	0.25	* 0.48	42.8	1.90
9.75	22.88	20.57	27.9	0.31	* 0.61	55.2	1.98
10	23.00	20.69	29.7	0.32	0.62	56.5	1.90
11	23.50	21.19	38.2	0.41	0.62	57.8	1.51
12	24.00	21.69	47.9	0.50	0.62	59.2	1.24
13	24.50	22.19	51.4	0.52	0.62	60.6	1.18

\*  $A_s$  shown reflects partially developed Bar F with 0% development at  $y_{stem} = 8.00 \text{ ft}$ .

**Q. Wall Design – Investigate Service Limit State [5.7.3.4]**

To ensure that the primary reinforcement is well distributed, crack control provisions are checked. They are dependent on the tensile stress in steel reinforcement at the service limit state, the concrete cover, and the geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level ( $\beta_s$ ). The Class 1 exposure factor is used ( $\gamma_e=1.0$ ) since the back face of the stem is not exposed once constructed.

The reinforcement spacing must satisfy

$$s \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Solve the equation above for the allowable reinforcement stress,  $f_{ssa}$ :

$$f_{ssa} = \frac{700 \cdot \gamma_e}{\beta_s \cdot (s + 2 \cdot d_c)} \leq 0.6 \cdot f_y$$

At  $y_{stem} = 13$  ft, Bars E and F are each spaced at 12" and alternated, providing #5 bars @ 6",  $A_s = 0.62$  in<sup>2</sup>/ft:

$$d_c = 2.00 + \frac{0.625}{2} = 2.31 \text{ in}$$

$$s = 6 \text{ in}$$

The strain ratio,  $\beta_s$ , is defined as:

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.31}{0.7 \cdot (24.5 - 2.31)} = 1.15$$

The allowable stress,  $f_{ssa}$  is:

$$f_{ssa} = \frac{700 \cdot \gamma_e}{\beta_s \cdot (s + 2 \cdot d_c)} = \frac{700 \cdot 1.0}{1.15 \cdot (6 + 2 \cdot 2.31)} = 57.3 \text{ ksi}$$

$$\text{or } f_{ssa} = 0.6 \cdot f_y = 0.6 \cdot 60 = 36.0 \text{ ksi} \quad \underline{\text{GOVERNS}}$$

[5.4.2.4]  
[5.7.1]

Find the actual stress provided in the steel:

The transformed area of reinforcement is:

$$n \cdot A_s = 8 \cdot (0.62) = 4.96 \text{ in}^2$$

Determine location of the neutral axis:

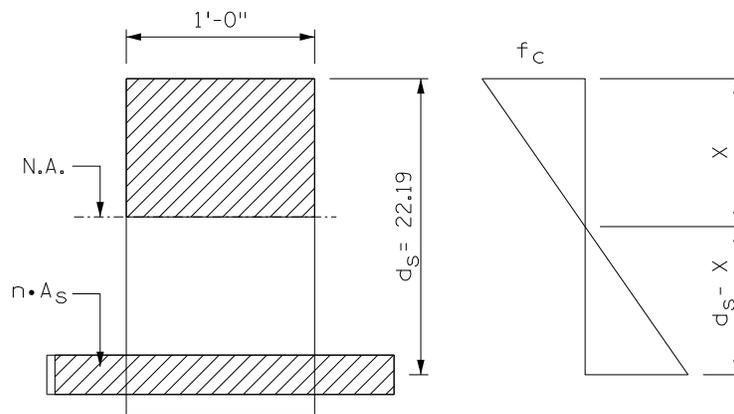


Figure 11.4.2.7

$$\frac{1}{2} \cdot b \cdot x^2 = n \cdot A_s (d_s - x)$$

$$\frac{1}{2} \cdot (12) \cdot x^2 = 4.96 (22.19 - x) \quad \text{solving, } x = 3.89 \text{ inches}$$

Then:

$$J \cdot d_s = d_s - \frac{x}{3} = 22.19 - \frac{3.89}{3} = 20.89 \text{ in}$$

$$\text{Actual } f_{ss} = \frac{M_{\text{service}}}{A_s \cdot j \cdot d_s} = \frac{30.4 \cdot 12}{0.62 \cdot (20.89)}$$

$$= 28.17 \text{ ksi} < 36.0 \text{ ksi}$$

OK

Table 11.4.2.16 summarizes the crack control check for the stem back face reinforcement over the height of the stem starting at  $y_{\text{stem}} = 8 \text{ ft}$ .

**Table 11.4.2.16 Crack Control Check**

$y_{\text{stem}}$ (ft)	$M_{\text{service}}$ (kip-ft)	Actual $A_s$ (in <sup>2</sup> /ft)	Bar spacing (in)	$d_s$ (in)	$\beta_s$	$f_{\text{ssa}}$ (ksi)	$x$ (in)	$j \cdot d_s$ (in)	Actual $f_{\text{ss}}$ (ksi)	C/D
8	8.4	0.31	12.0	19.69	1.17	36.0	2.65	18.81	17.29	2.08
9	11.3	* 0.48	12.0	20.19	1.16	36.0	3.29	19.09	14.80	2.43
9.75	14.1	* 0.61	12.0	20.57	1.16	36.0	3.70	19.34	14.34	2.51
10	15.0	0.62	6.0	20.69	1.16	36.0	3.74	19.44	14.93	2.41
11	19.4	0.62	6.0	21.19	1.16	36.0	3.79	19.93	18.84	1.91
12	24.5	0.62	6.0	21.69	1.15	36.0	3.84	20.41	23.23	1.55
13	30.4	0.62	6.0	22.19	1.15	36.0	3.89	20.89	28.17	1.28

\*  $A_s$  shown reflects partially developed Bar F with 0% development at  $y_{\text{stem}} = 8.00 \text{ ft}$ .

**O. Wall Stem  
Design –  
Investigate Shear**

Shear typically does not govern the design of retaining walls. If shear does become an issue, the thickness of the stem should be increased such that transverse reinforcement is not required. Calculations will be shown for the shear check at the bottom of the stem. Shear checks at other locations are summarized in Table 11.4.2.17

Conservatively ignoring the benefits of axial compression and the shear key, the shear capacity of the stem can be shown to be greater than that required.

[5.8.3.3-1]

$$V_n = V_c + V_s + V_p$$

Recognizing that  $V_s$  and  $V_p$  are zero,

$$V_n = V_c$$

[5.8.3.3-3]

$$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

[5.8.3.4.2]

[5.8.2.9]

At the bottom of the stem, the area of reinforcement is 0.62 in<sup>2</sup>. The effective shear depth,  $d_v$ , is calculated as follows:

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.62 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.91 \text{ in}$$

For #5 back face bars,  $d = 24.5 - 2 - 0.625/2 = 22.19 \text{ in}$

$$d_v \text{ is the max of: } 0.72 \cdot h = 0.72 \cdot 24.5 = 17.64 \text{ in}$$

$$0.9 \cdot d = 0.9 \cdot 22.19 = 19.97 \text{ in}$$

$$d - a/2 = 22.19 - 0.91/2 = 21.74 \text{ in} \quad \underline{\text{GOVERNS}}$$

$\beta$  will be calculated using the Sectional Design Model of 5.8.3.4.2.

The crack spacing parameter,  $s_{xe}$ , is taken as:

$$s_{xe} = s_x \cdot \frac{1.38}{a_g + 0.63} \quad \text{and} \quad 12 \text{ in} \leq s_{xe} \leq 80 \text{ in}$$

$$s_x = d_v = 21.74 \text{ in}$$

$$a_g = 0.75 \text{ in (assumed)}$$

Then

$$s_{xe} = 21.74 \cdot \frac{1.38}{0.75 + 0.63} = 21.74 \text{ in}$$

$$\epsilon_s = \frac{\left( \frac{|M_u|}{d_v} + 0.5 \cdot N_u + |V_u| \right)}{E_s \cdot A_s} = \frac{\left( \frac{|44.5 \cdot 12|}{21.74} + 0.5 \cdot 0 + 9.0 \right)}{29000 \cdot 0.62} = 0.00187$$

Where the magnitude of the moment,  $M_u$ , is not to be less than:

$$M_u \geq V_u \cdot d_v \geq 9.0 \cdot 21.74 \cdot 1/12 \geq = 16.3 \text{ k-ft} \quad \underline{\text{OK}}$$

Because there is no shear reinforcement the value of  $\beta$  is taken as:

$$\beta = \frac{4.8}{1 + 750 \cdot \epsilon_s} \cdot \frac{51}{39 + s_{xe}} = \frac{4.8}{1 + 750 \cdot 0.00187} \cdot \frac{51}{39 + 21.74} = 1.68$$

The factored shear resistance at  $y_{stem} = 13$  is then:

$$\phi V_c = 0.9 \cdot 0.0316 \cdot 1.68 \cdot \sqrt{4} \cdot 12 \cdot 21.74$$

$$= 24.9 \text{ kips} > V_u = 9.0 \text{ kips} \quad \underline{\text{OK}}$$

**Table 11.4.2.17 Stem Shear Check**

$y_{stem}$ (ft)	$V_u$ (kip)	Actual $A_s$ (in <sup>2</sup> /ft)	$M_u$ (kip-ft)	$d_v$ (in)	$V_u \cdot d_v$ (k-ft)	$\epsilon_s$	$\beta$	$\phi V_n$ (kip)	C/D
2	0.5	0.31	0.5	16.46	0.7	0.00011	4.08	45.8	91.6
4	1.4	0.31	2.2	17.46	2.0	0.00032	3.50	41.7	29.8
6	2.5	0.31	6.0	18.46	3.8	0.00071	2.78	35.0	14.0
8	4.0	0.31	12.7	19.46	6.5	0.00132	2.10	27.9	6.98
9	4.8	0.31	16.9	19.96	8.0	0.00166	1.85	25.2	5.25
9.75	5.5	0.31	21.0	20.34	9.3	0.00199	1.66	23.0	4.18
10	5.8	* 0.35	22.3	20.44	9.9	0.00186	1.72	24.0	4.14
11	6.8	* 0.53	28.7	20.80	11.8	0.00152	1.91	27.1	3.99
12	7.9	0.62	36.0	21.24	14.0	0.00157	1.87	27.1	3.43
13	9.0	0.62	44.5	21.74	16.3	0.00187	1.68	24.9	2.77

\*  $A_s$  shown reflects partially developed Bar F with 0% development at  $y_{stem} = 9.75$  ft.

**R. Design Front Face Vertical Reinforcement**

The front face vertical reinforcement consists of Bar G lapped with Bar D. The LRFD standards examined wind loading in a construction limit state. This was envisioned as a point in construction where winds were high on the freestanding stem without backfill. In this state, the front face reinforcement is in tension and the concrete strength will have only achieved half its strength at the time of form removal.

For a wall with a 13 foot stem height, a #5 Bar D and a #4 Bar G spaced at 12 inches were found adequate for the design. With taller wall heights, there were cases where this did not meet the flexural demand. In those cases, a column design run was performed treating the stem as a doubly-reinforced section with an axial load equal to 90% of the stem self-weight. Using these assumptions, Bar G met the flexural and shear demands with a #4 size.

**S. Design Stem Wall Shrinkage and Temperature Reinforcement [5.10.8]**

To ensure good performance, a minimum amount of reinforcement needs to be placed near each face of concrete elements. This reinforcement limits the size of cracks associated with concrete shrinkage and temperature changes. Since the wall thickness varies, the average thickness was used.

$$A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.30 \cdot 13 \cdot 12 \cdot 21.25}{2 \cdot (13 \cdot 12 + 21.25) \cdot 60} = 0.20 \text{ in}^2/\text{ft}$$

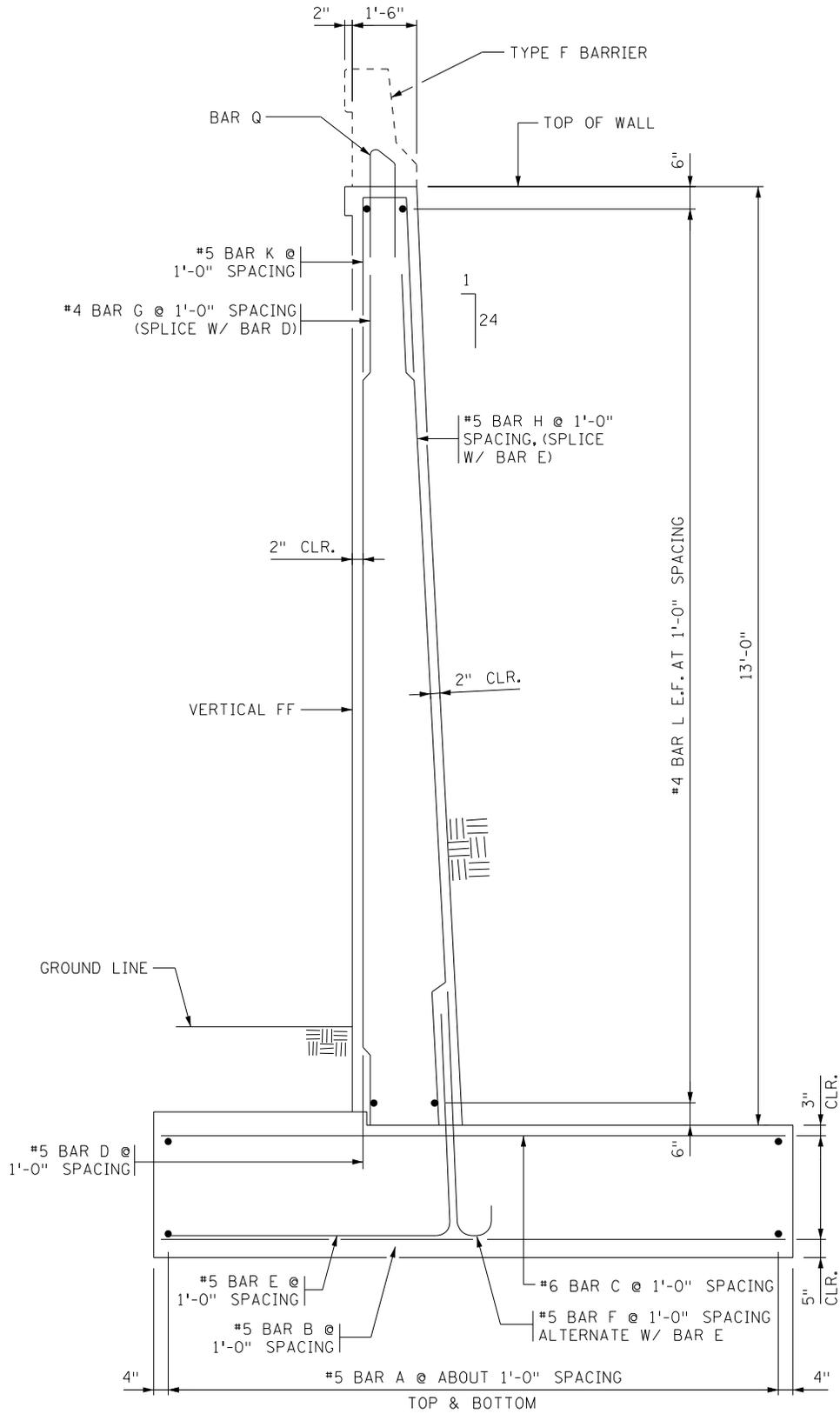
$$0.11 \leq A_s \leq 0.60$$

This is the minimum area of reinforcement that must be placed in each direction and on each face.

Use #4 @ 12" ( $A_s = 0.20 \text{ in}^2/\text{ft}$ ) for stem wall front back face horizontal bars (Bar L).

***T. Summary***

The wall section shown in Figure 11.4.2.8 summarizes the design of the retaining wall. Note that the spacing of the longitudinal footing bars is revised slightly from previous calculations for detailing purposes.

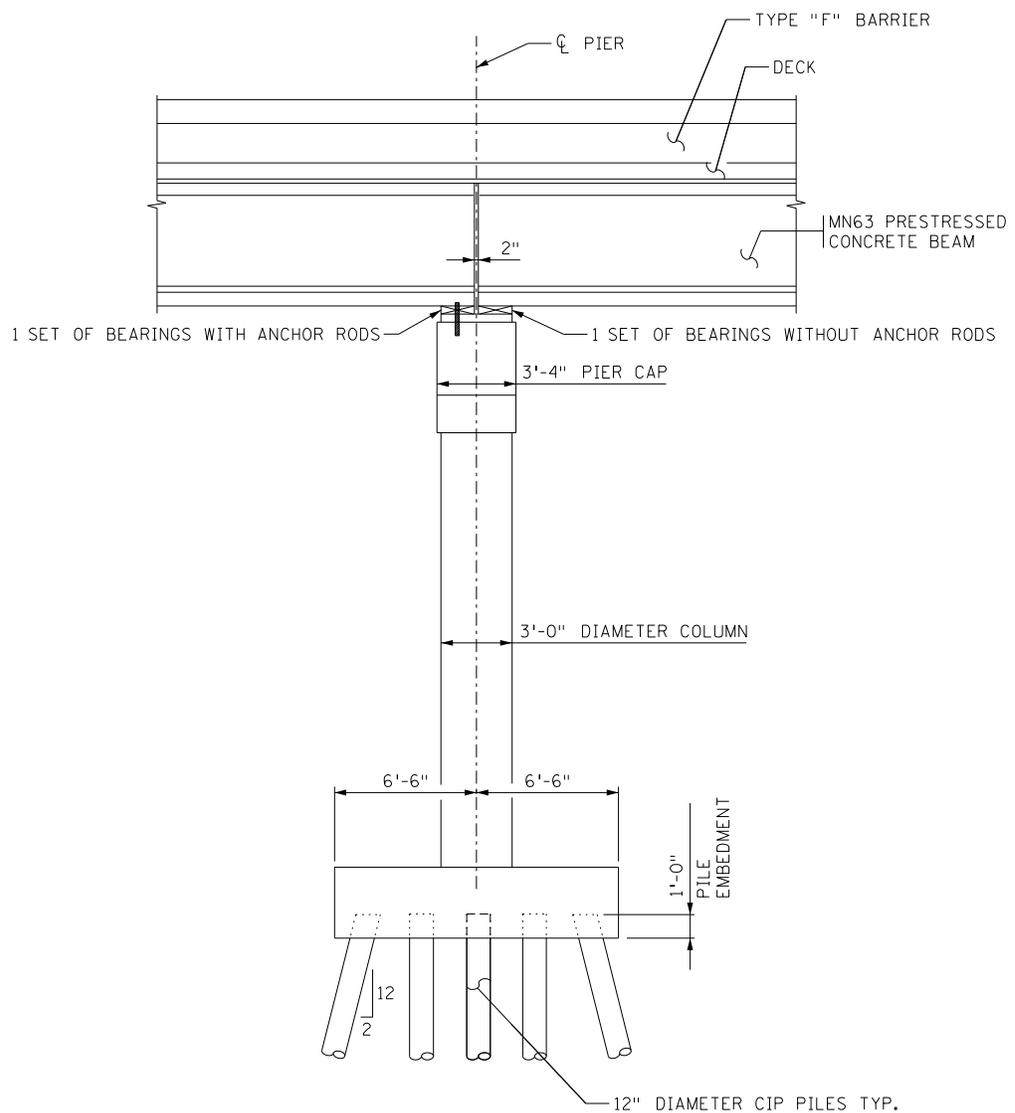


**Retaining Wall Design Summary**  
**Figure 11.4.2.8**

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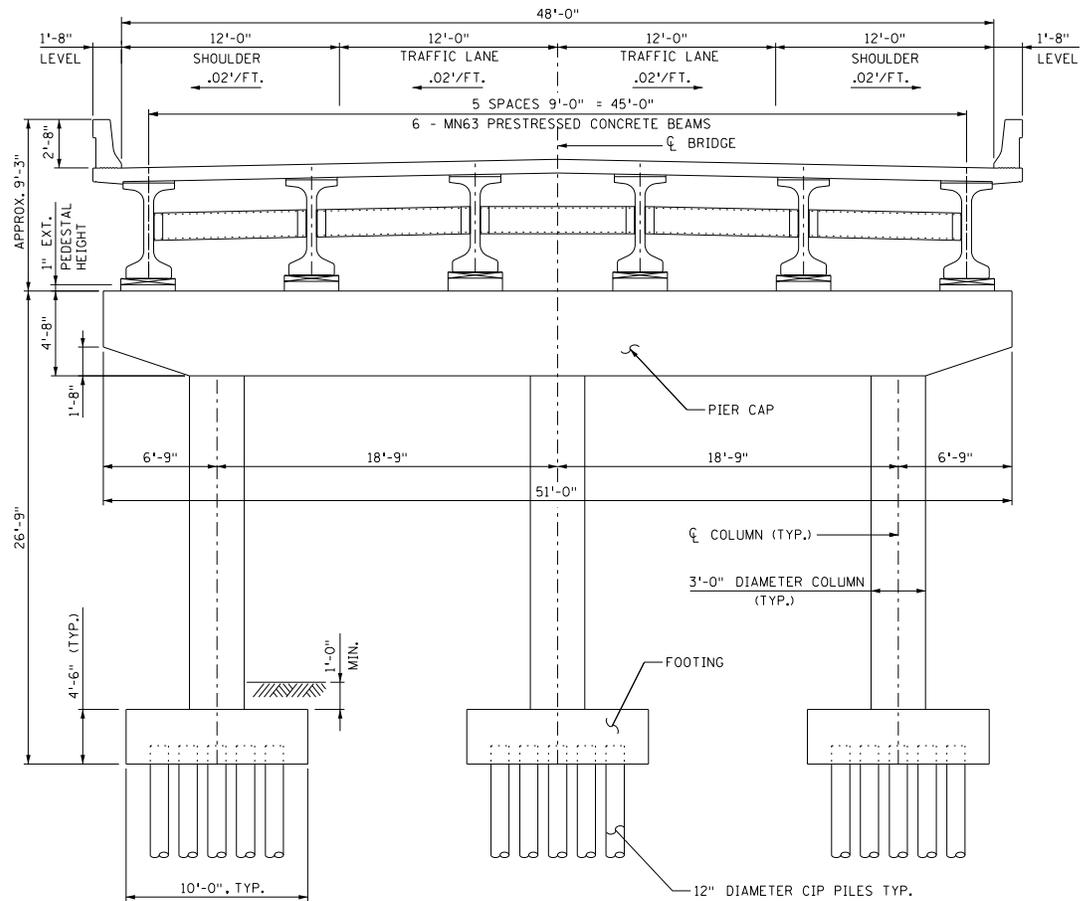
**11.4.3 Three-Column Pier Design Example**

This example illustrates the design of a reinforced concrete three-column pier. The bridge carries a two-way roadway consisting of one 12'-0" traffic lane and one 12'-0" shoulder in each direction. The superstructure has two equal spans of 130'-0" consisting of a 9" deck supported by 63" deep prestressed beams spaced 9'-0" on center. The bridge has a Type "F" barrier (Fig. 5-397.115) on each side of the bridge deck and diaphragms are provided at the supports and at the interior third points. The superstructure is part of a grade-separation structure and is considered translationally fixed at the pier. The bearings are curved plate bearings (3¼" in height, see Bridge Details B310). An end view of the pier is presented in Figure 11.4.3.1. Two sets of bearings rest on the pier cap, one set for the beams of each span. To simplify design, only one reaction is used per beam line, acting at the centerline of pier.



**Figure 11.4.3.1**

The pier cap is supported by three columns. The columns are supported by separate pile foundations. An elevation view of the pier is presented in Figure 11.4.3.2.



**Figure 11.4.3.2**  
**3-Column Pier - Elevation**

Pier design is accomplished with a top down approach. The design parameters and loads are determined first - followed by the pier cap, column, and footing designs.

The following terms are used to describe the orientation of the structural components and loads. The terms "longitudinal" and "transverse" are used to describe global orientation relative to the superstructure and roadway. The terms "parallel" and "perpendicular" are used to define the orientations relative to the pier. The parallel dimension is the "long" direction of the structural component and the perpendicular dimension is 90° to the parallel dimension and is in the direction of the "short" side. The distinction becomes clear in describing the load path for lateral forces applied to bridges with substructures skewed to the superstructure.

Forces parallel and perpendicular to the pier arise from combining the component forces applied transversely and longitudinally to the superstructure. The pier for this example is not skewed, consequently transverse forces are equivalent to parallel pier forces. However, to ensure the clarity of future designs, the parallel and perpendicular nomenclature will be used.

### A. Material and Design Parameters

#### Pier Cap

The cap must have sufficient length to support all of the beam lines and their bearings. It also must have sufficient width to support two lines of bearings and provide adequate edge distances for the bearings. Pedestals are constructed on the pier cap to accommodate the different heights at which the prestressed beams are supported due to the cross slope of the deck. When beginning a design, first determine the required width and then try a cap depth equal to 1.4 to 1.5 times the width.

**Table 11.4.3.1 – Pier Cap Parameters**

Parameter	Label	Value
Width of Pier Cap	$b_{cap}$	40 in
Length of Pier Cap	$L_{cap}$	51 ft = 612 in
Depth of Pier Cap at Center	$d_{mid}$	56 in
Depth of Pier Cap at Ends	$d_{end}$	36 in

#### Columns

In order to avoid interference between the column vertical bars and pier cap reinforcement, choose columns with a diameter slightly smaller than the width of the pier cap. Columns should also be proportioned relative to the depth of the superstructure. For 63" prestressed beams a column diameter of at least 36 inches should be used. (See Section 11.2.1.)

**Table 11.4.3.2 – Column Parameters**

Parameter	Label	Value
Column Diameter	$d_{col}$	36 in
Number of Columns	$N_{col}$	3
Column Cross-Sectional Area	$A_g$	$\frac{\pi \cdot 36^2}{4} = 1018 \text{ in}^2$
Column Moment of Inertia	$I_g$	$\frac{\pi \cdot 36^4}{64} = 82,450 \text{ in}^4$

#### Footings and Piles

A rectangular footing with the following properties will be tried initially:

**Table 11.4.3.3 – Foundation Parameters**

Parameter	Label	Value
Pile Type	-	Cast-In-Place
Pile Diameter	$d_{pile}$	12 in
Depth of Footing	$d_{foot}$	4.50 ft
Width of Footing Parallel to Pier	$b_{foot}$	10.0 ft
Length of Footing Perpendicular to Pier	$L_{foot}$	13.0 ft

Use the Bridge Construction Unit’s Foundation Recommendations to identify the pile design capacity  $R_N$  and resistance factor  $\phi_{dyn}$  to be used:

Nominal Capacity  $R_N = 200$  tons/pile

Resistance Factor  $\phi_{dyn} = 0.50$

Bearing Resistance  $R_f = 0.50 \cdot 200 = 100$  tons/pile = 200 kips/pile

**Location of Columns**

The outside columns should be positioned to minimize dead load moments in the columns and also balance the negative moments in the pier cap over the columns. A rule of thumb is to use an overhang dimension (measured from edge of outside column to centerline of exterior beam) equal to  $\frac{1}{5}$  of the column spacing. After trying several layouts, outside columns located 18.75 feet from the center of the bridge were found to minimize design forces.

The following material weights and strengths are used in this example:

**Table 11.4.3.4 – Unit Weights and Strengths**

Parameter	Label	Value
Unit Weight of Concrete	$\gamma_C$	0.145 kcf (strength) 0.150 kcf (loads)
Concrete Compressive Strength	$f'_c$	4 ksi
Modulus of Elasticity, Concrete	$E_c$	$33,000 \cdot (0.145)^{1.5} \cdot \sqrt{4}$ = 3644 ksi
Yield Strength of Reinforcement	$f_y$	60 ksi
Modulus of Elasticity, Reinforcement	$E_s$	29,000 ksi
Modular Ratio	$n$	8
Soil Unit Weight	$\gamma_{soil}$	0.120 kcf

**B. Determine Design Loads**

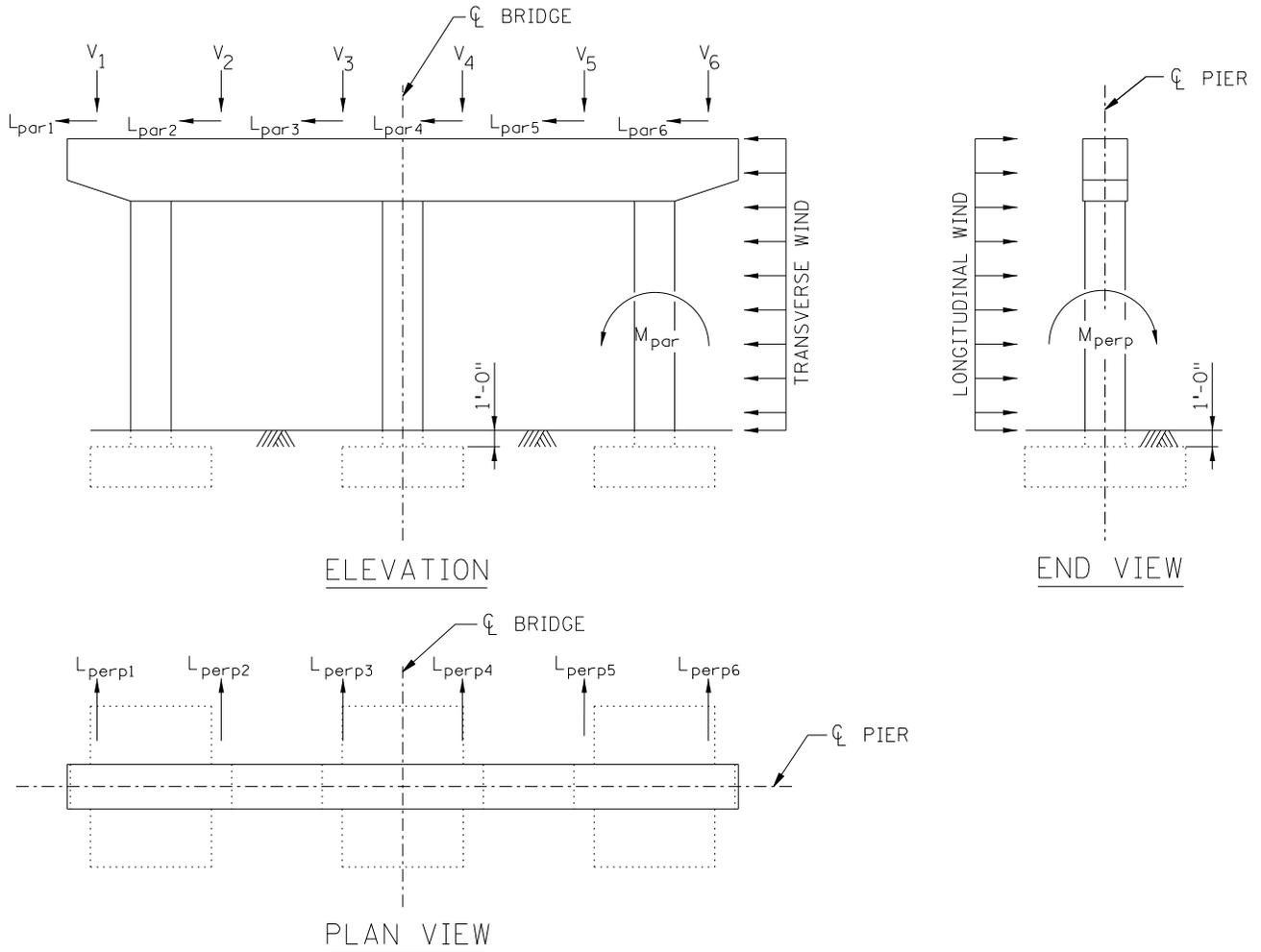
The loads applied to the three-column pier include dead load, live load, braking force, wind on structure, wind on live load, and uniform temperature change. The pier is located more than 30 feet from the

edge of the travel lane, so vehicular collision forces will not be considered.

#### **Application of Loads to the Structural Model**

Aside from wind on substructure and internal temperature change forces, the loads applied to the pier are transferred from the superstructure to the pier cap via the bearings. Figure 11.4.3.3 illustrates the load components that are transferred from the bearings to the pier cap. At each girder location three load components are possible, a parallel force, a perpendicular force, and a vertical force. In the following load tables, vertical force components are identified as  $V_1$  to  $V_6$ . Parallel forces have labels of  $L_{Par1}$  to  $L_{Par6}$ , and perpendicular forces are identified as  $L_{Perp1}$  to  $L_{Perp6}$ .

For several loads applied to the pier, the concrete deck was assumed to be a rigid diaphragm. A rigid deck assumption combined with the presence of diaphragms at the pier permits one to assume that the parallel and perpendicular wind loads can be evenly distributed among the bearings. Varying vertical reactions resist lateral and vertical loads that produce an overturning moment.



**Figure 11.4.3.3**  
**Loads Applied to the Pier**

The superstructure dead loads applied to the pier consist of the following: the design shear in the prestressed beam at the centerline of bearing, the beam ends (portion of the beams beyond centerline of bearing), the portion of deck, stool, barrier, and future wearing course between centerline of bearings, the cross-frames at the pier, two sets of bearings per beam line, and the pedestals. (For this example, pedestals are considered part of the superstructure for load calculations.)

Assume the following for dead load calculations:

- a concrete stool height of 2.5 inches
- a pedestal size of 36 inches (perpendicular to pier) x 44 inches (parallel to pier) with average height of 3.5 inches
- 2 lines of interior diaphragms in each span
- 1 line of pier diaphragms in each span

Table 11.4.3.5 summarizes the superstructure dead loads.

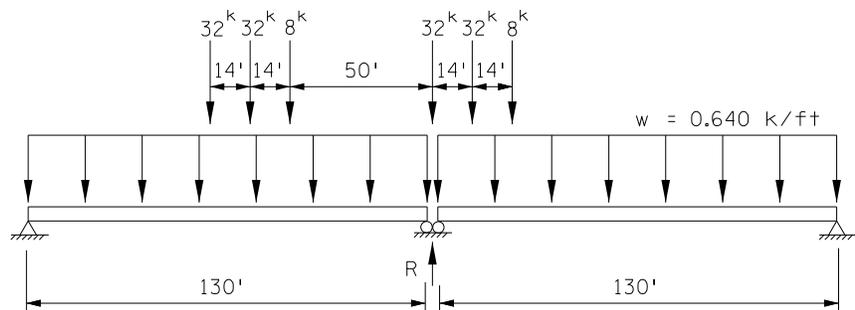
**Table 11.4.3.5 - Superstructure Dead Loads (kips)**

Load	V1	V2	V3	V4	V5	V6
MN63 Beams	113.0	113.0	113.0	113.0	113.0	113.0
Interior Diaphragms	0.4	0.9	0.9	0.9	0.9	0.4
Pier Diaphragms	0.4	0.9	0.9	0.9	0.9	0.4
Deck	117.0	131.6	131.6	131.6	131.6	117.0
Stool	11.5	11.5	11.5	11.5	11.5	11.5
F-Barriers	20.1	20.1	20.1	20.1	20.1	20.1
Future Wearing Course (FWC)	20.8	20.8	20.8	20.8	20.8	20.8
Additional DC beyond the centerlines of bearing due to beams, deck, stool, barriers, and FWC	2.9	3.1	3.1	3.1	3.1	2.9
Bearings	0.5	0.5	0.5	0.5	0.5	0.5
Pedestals	0.7	0.7	0.7	0.7	0.7	0.7
Total	287.3	303.1	303.1	303.1	303.1	287.3

**[3.6.1]**

**Live Load**

First, the maximum reaction at the pier due to a single lane of HL-93 live load must be determined. After comparing results from several configurations, the double truck with lane load shown in Figure 11.4.3.4 was found to produce the largest reaction. For simply supported superstructures, the 0.9 multiplier per AASHTO Article 3.6.1.3.1 is used.



**Figure 11.4.3.4**  
**Live Load Configuration For Maximum Pier Reaction**

**[3.6.1.3.1]**

Table 11.4.3.6 lists the live load reactions at the pier for different numbers of lanes loaded. It also includes the maximum reaction for fatigue, which occurs when the center axle of the fatigue truck is directly over the pier. Note that only 1 lane is used for the fatigue truck reaction calculation.

**[3.6.1.4.1]**

**Table 11.4.3.6 – Live Load Reactions on Pier (per lane)**

Loading	Truck Load Reaction with Dynamic Load Allowance (kips)	Lane Load Reaction (kips)	Product of Multiple Presence Factor and Double Truck Load Factors	Total Reaction R (kips)	Uniform Load $w = R/10'$ (kips/ft)
1 Lane	134.1 - Double Truck	83.2	$1.20 \cdot 0.90 = 1.080$	234.7	23.5
2 Lanes	134.1 - Double Truck	83.2	$1.00 \cdot 0.90 = 0.900$	195.6	19.6
3 Lanes	134.1 - Double Truck	83.2	$0.85 \cdot 0.90 = 0.765$	166.2	16.6
4 Lanes	134.1 - Double Truck	83.2	$0.65 \cdot 0.90 = 0.585$	127.1	12.7
Fatigue	73.3 - Fatigue Truck	0.0	1.00	73.3	7.3

The next step is to determine the live load cases that will produce the maximum force effects in the cap, columns, and foundation of the pier. This is done by positioning the single lane reactions in lanes across the transverse bridge cross-section to get the desired effect.

For instance, to obtain the maximum positive moment in the pier cap, place one or two live load lane reactions on the deck such that the beams located between the columns receive the maximum load. Figure 11.4.3.5 illustrates the live load cases used in the example. Table 11.4.3.7 contains beam reactions for each of the load cases. Load distribution for determination of values in the table is based on assuming simple supports at each beam.

For example, for Live Load Case 2:

$$w = 23.5 \text{ kips/ft}$$

$$V_1 = V_6 = 0$$

$$V_2 = V_5 = 23.5 \cdot \frac{(9 - 8.50)^2}{2} \cdot \left(\frac{1}{9}\right) = 0.3 \text{ kips}$$

$$V_3 = V_4 = 23.5 \cdot 9 \cdot \frac{1}{2} + 23.5 \cdot 0.5 \cdot 8.75 \cdot \frac{1}{9} = 117.2 \text{ kips}$$

Other live load cases with slight variations in live load placement might be found that will result in greater load effects to the pier cap and columns, but the increase in magnitude is relatively small or does not govern the design and therefore has not been included in this example. For instance, only one case for 4 live load lanes was included in the check

for this example. Two other cases could be considered in place of Live Load Case 8. One case would consist of the two middle lanes abutting each other with a 2'-0" gap between the center and outside lanes. The other case would include a 2'-0" gap between the two middle lanes, and separate the outside and middle lanes by 1'-0". The designer is responsible for investigating all load cases that may affect the design.

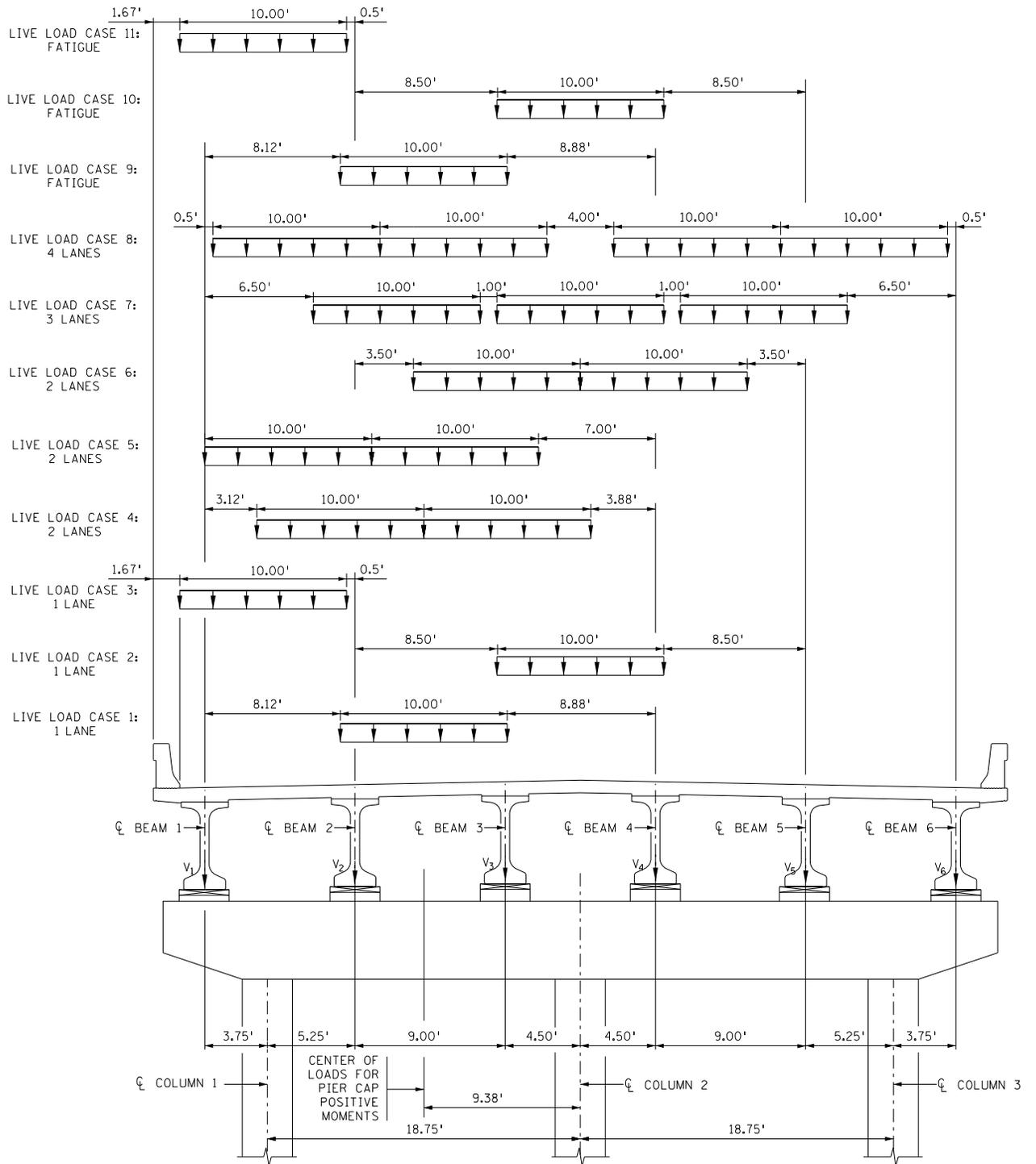


Figure 11.4.3.5

**Table 11.4.3.7 - Superstructure Live Load Beam Reactions (kips)  
(includes dynamic load allowance)**

Live Load Case	Location	V1	V2	V3	V4	V5	V6
1	One Lane Positive Cap Moment	1.0	125.4	108.6	0.0	0.0	0.0
2	One Lane Over Center Column	0.0	0.3	117.2	117.2	0.3	0.0
3	One Lane At Gutter Line	143.6	94.3	0.0	0.0	0.0	0.0
4	Two Lanes At Cap Midspan Between Columns - Positive Cap Moment	37.6	165.8	160.0	28.5	0.0	0.0
5	Two Lanes Max Load to Beam 2 - Positive Cap Moment	88.2	176.4	123.0	4.4	0.0	0.0
6	Two Lanes Over Center Column	0.0	32.9	163.1	163.1	32.9	0.0
7	Three Lanes Over Center Column	5.8	108.6	134.6	134.6	108.6	5.8
8	Four Lanes	51.0	114.1	88.9	88.9	114.1	51.0
9	Fatigue-One Lane Positive Cap Moment	0.3	39.0	33.7	0.0	0.0	0.0
10	Fatigue-One Lane Over Center Column	0.0	0.1	36.4	36.4	0.1	0.0
11	Fatigue - One lane At Gutter Line	44.6	29.3	0.0	0.0	0.0	0.0

**Braking Force**

For this example, 4 design lanes will fit on the bridge, but it is assumed the bridge will at most see 2 lanes loaded in one direction for braking in the future. The 2 lanes of traffic are assumed to transmit a longitudinal (perpendicular to the pier) force that is evenly distributed to the six bearings and three columns.

**[3.6.4]**

Begin by determining if a truck by itself or if truck plus lane loading governs the braking force.

Truck alone:

$$0.25 \cdot (8 + 32 + 32) = 18.0 \text{ kips} \quad \text{GOVERNS}$$

Truck plus lane:

$$0.05 \cdot [8 + 32 + 32 + (2 \cdot 130 \cdot 0.64)] = 11.9 \text{ kips}$$

Then the design force is:

$$BR = 18.0 \cdot (\# \text{ of lanes in one direction}) \cdot (\text{multiple presence factor})$$

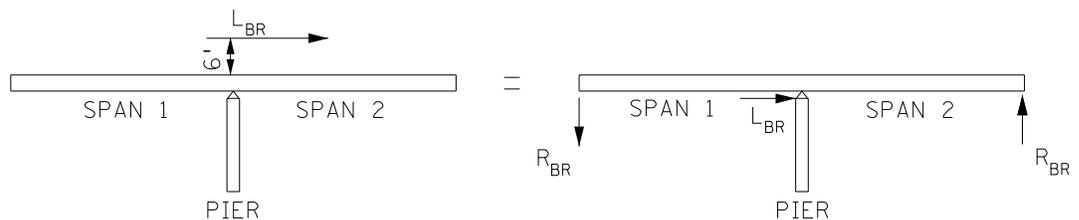
$$= 18.0 \cdot 2 \cdot 1.0 = 36.0 \text{ kips}$$

Although the lateral braking force is to be applied 6 feet above the top of deck, it gets transferred to the pier through the bearings. For a description of how the load is applied to the analysis model, see Article 11.4.3C in this example. In order to satisfy statics and make the two

load systems equivalent, transfer of the lateral force down to the bearing level requires the addition of a moment couple equal to:

$$L_{BR} \cdot [6 \text{ ft} + (\text{distance from top of deck to bearings})]$$

Figure 11.4.3.6 illustrates this. The moment couple consists of vertical forces at the abutments. Because the distance from abutment to abutment is very large relative to the transfer height, the vertical forces are negligible and will be ignored. Therefore, we can conclude that for pier analysis, the braking force can be applied at the top of the pier. Also, the bearings allow for rotation due to longitudinally applied loads. This prevents the moment from transferring to the pier even when the load is applied above the top of the pier.



**Figure 11.4.3.6**  
**Equivalent Load Systems**

Height of load application  $y_{BR}$  above the top of footing is:

$$y_{BR} = 26.75 - 4.50 = 22.25 \text{ ft}$$

The moment at the base of the columns is:

$$M_{\text{perpBR}} = 36.0 \cdot 22.25 \cdot \left(\frac{1}{3}\right) = 267.0 \text{ kip-ft/column}$$

The lateral load on each bearing is:

$$L_{BR} = 36.0 \cdot \left(\frac{1}{6}\right) = 6.0 \text{ kip/bearing}$$

**[3.8.1.2.3]**

**Wind Loads**

Wind loads consist of the transverse and longitudinal wind load components transmitted by the superstructure to the substructure and the wind load applied directly to the substructure. The wind load is applied for various angles of wind direction and is taken as the product of the skew coefficients, the calculated wind pressure, and the depth of the bridge. For this design example, wind loads are determined for the

Strength III and Strength V load combinations; however, calculations are only shown for Strength III.

The wind pressure for Strength III is determined as:

**[3.8.1.2.1]**

$$P_{zIII} = 2.56 \cdot 10^{-6} \cdot V^2 \cdot K_z \cdot G \cdot C_D$$

The design 3-second gust wind speed is determined using Figure 3.8.1.1.2-1 as  $V = 115$  mph.

Wind exposure category C is assumed for the structure and, with a superstructure height  $< 33$  ft, the pressure exposure and elevation coefficient,  $K_z$ , is taken as 1.0.

**[Table C3.8.1.2.1-1]**

**[Table 3.8.1.2.1-1]**

**[Table 3.8.1.2.1-2]**

The gust effect factor,  $G$ , is taken as 1.0. The drag coefficient,  $C_D$ , is:

- 1.3 for wind on the superstructure
- 1.6 for wind on the substructure

Therefore, the wind pressure for Strength III with wind exposure category C is:

$$P_{zsupIII} = 2.56 \cdot 10^{-6} \cdot 115^2 \cdot 1.0 \cdot 1.0 \cdot 1.3 = 0.044 \text{ ksf on superstructure}$$

$$P_{zsubIII} = 2.56 \cdot 10^{-6} \cdot 115^2 \cdot 1.0 \cdot 1.0 \cdot 1.6 = 0.054 \text{ ksf on substructure}$$

The wind pressure on the structure for Strength V is based on a design 3-second gust wind speed  $V = 80$  mph, and is taken as:

**[Table C3.8.1.2.1-2]**

$$P_{zsupV} = 0.0163 \cdot C_D = 0.0163 \cdot 1.3 = 0.021 \text{ ksf on superstructure}$$

$$P_{zsubV} = 0.0163 \cdot C_D = 0.0163 \cdot 1.6 = 0.026 \text{ ksf on substructure}$$

**[3.8.1.2.3a]**

**Wind Load from the Superstructure**

For the transverse wind load on the superstructure, the deck functions as a horizontal 2-span continuous beam with wind pressure acting on the exposed edge area of the superstructure. The reaction at the fixed end for a propped cantilever beam is  $\frac{5}{8}$  of the uniformly applied load. Then for a 2-span continuous beam,  $\frac{5}{8}$  of wind from both spans is carried by the pier.

Assuming a fixed bearing assembly height of 3.25 inches, the depth of exposure  $D_{wexp}$  is approximately:

$$\begin{aligned} D_{wexp} &= \text{barrier height} + \text{deck thickness} + \text{stool height} \\ &\quad + \text{beam depth} + \text{bearing assembly height} \\ &\quad + \text{exterior pedestal height} - (\text{cross-slope drop}) \\ &= 32.00 + 9.00 + 2.50 + 63.00 + 3.25 + 3.00 - (0.02 \cdot 18.00) \\ &= 112.39 \text{ in} \end{aligned}$$

Round up to  $113'' = 9'-5'' = 9.42 \text{ ft}$

Then the tributary area for superstructure wind is:

$$A_{wsup} = 2 \cdot \left(\frac{5}{8}\right) \cdot 130 \cdot 9.42 = 1531 \text{ ft}^2$$

For the longitudinal wind load on the superstructure, the deck does not function in the same way. All of the longitudinal wind load could be applied to the pier. However, some of the load is taken at the abutments due to friction in the bearings. Therefore, 5/8 is considered a reasonable approximation of the longitudinal component applied to the pier.

The wind on superstructure load  $WS_{sup}$  is:

$$WS_{sup} = P_B \cdot A_{wsup}$$

$$\text{where } P_B = P_Z \cdot \text{Skew Coefficient}$$

The skew coefficients are taken from LRFD Table 3.8.1.2.3a-1 for various attack angles.

For example, for Strength III and a wind attack angle skewed 30 degrees to the superstructure:

$$P_{Btransv} = P_{Bpar} = 0.044 \cdot 0.82 = 0.036 \text{ ksf}$$

$$P_{Blong} = P_{Bperp} = 0.044 \cdot 0.24 = 0.011 \text{ ksf}$$

$$WS_{suppar} = 0.036 \cdot 1531 = 55.1 \text{ kips}$$

$$L_{par1} = L_{par2} = L_{par3} = L_{par4} = L_{par5} = L_{par6} = \frac{55.1}{6} = 9.2 \text{ kips}$$

$$WS_{supperp} = 0.011 \cdot 1531 = 16.8 \text{ kips}$$

The longitudinal and transverse wind components are applied at the mid-depth of the superstructure.

Similar to the braking force, the longitudinal wind component on the superstructure can be applied at the top of the pier for analysis. The height of application  $y_{perp}$  above the top of footing is:

$$y_{perp} = 26.75 - 4.50 = 22.25 \text{ ft}$$

Then, for a wind attack angle skewed 30 degrees to the superstructure, the moment at the base of the columns is:

$$M_{\text{wsupperp}} = 16.8 \cdot 22.25 \cdot \left(\frac{1}{3}\right) = 124.6 \text{ kip} - \text{ft/column}$$

Because the bearings do not rotate due to transverse loads, the moment does not dissipate at the bearings and needs to be applied to the pier. For the analysis model, the transverse wind component will be applied at the centroid of the pier cap. Transfer of the transverse wind component from the centroid of the exposed superstructure area to the centroid of the pier cap requires the addition of vertical loads at the bearings equivalent to the reduction in moment  $M_{\text{red}}$ . For a wind attack angle skewed 30 degrees:

$$\begin{aligned} M_{\text{red}} &= WS_{\text{suppar}} \cdot (\text{distance from superstructure centroid to pier cap centroid}) \\ &= 55.1 \cdot \left(\frac{9.42}{2} + \frac{4.67}{2}\right) = 388.2 \text{ kip} - \text{ft} \end{aligned}$$

The additional vertical loads are calculated assuming the moment is applied at the center of the bridge and the deck is rigid. The "I" of the beams is determined and vertical loads "V" are based on the formula:

$$V = \frac{M_{\text{red}} x_{\text{beam}}}{I_{\text{beams}}}$$

where  $x_{\text{beam}}$  = distance from center of bridge to centerline of beam  
and  $I_{\text{beams}} = \sum x_{\text{beam}}^2$

Then for Beam 1 (left fascia beam with vertical load  $V_1$ ) and a wind attack angle skewed 30 degrees:

$$x_{\text{beam1}} = 22.5 \text{ ft}$$

$$\begin{aligned} I_{\text{beams}} &= \sum x^2 = (22.5)^2 + (13.5)^2 + (4.5)^2 + (-4.5)^2 + (-13.5)^2 + (-22.5)^2 \\ &= 1417.5 \text{ ft}^2 \end{aligned}$$

$$V_1 = \frac{388.2 \cdot (22.5)}{1417.5} = 6.2 \text{ kips}$$

For Strength III, the wind on superstructure loads applied to the pier are summarized in Table 11.4.3.8.

**Table 11.4.3.8 – Wind Load from Superstructure (Strength III)**

Wind Attack Angle	L <sub>par1</sub> V <sub>1</sub> (kips)	L <sub>par2</sub> V <sub>2</sub> (kips)	L <sub>par3</sub> V <sub>3</sub> (kips)	L <sub>par4</sub> V <sub>4</sub> (kips)	L <sub>par5</sub> V <sub>5</sub> (kips)	L <sub>par6</sub> V <sub>6</sub> (kips)	M <sub>wsuperp</sub> (kip-ft/col)
0 Degree Skew	11.2 7.5	11.2 4.5	11.2 1.5	11.2 -1.5	11.2 -4.5	11.2 -7.5	0.0
15 Degree Skew	10.0 6.7	10.0 4.0	10.0 1.3	10.0 -1.3	10.0 -4.0	10.0 -6.7	57.1
30 Degree Skew	9.2 6.2	9.2 3.7	9.2 1.2	9.2 -1.2	9.2 -3.7	9.2 -6.2	124.6
45 Degree Skew	7.4 5.0	7.4 3.0	7.4 1.0	7.4 -1.0	7.4 -3.0	7.4 -5.0	158.7
60 Degree Skew	3.8 2.6	3.8 1.5	3.8 0.5	3.8 -0.5	3.8 -1.5	3.8 -2.6	192.8

**[3.8.2]**

**Vertical Wind Load**

An additional wind on superstructure load is considered for the case where there is no live load and the wind is oriented at 0 degrees. The load represents uplift on the bottom of the deck and is called the vertical wind on superstructure load. The vertical wind load case is applied to Strength III only. The deck was assumed hinged over the pier for this load case. A tributary length of 130 feet (2 · 0.5 · 130 feet) was used with the deck width of 51.33 feet. A 0.020 ksf pressure produces a vertical force of:

Vertical force:

$$WS_v = 51.33 \cdot -0.020 \cdot 130 = -133.5 \text{ kips}$$

Eccentricity of vertical force:

$$e_{wsv} = -\left(\frac{51.33}{4}\right) = -12.83 \text{ ft}$$

Overturning Moment:

$$M_{wsv} = -133.5 \cdot (-12.83) = 1713 \text{ k-ft}$$

The vertical force applied to the pier at each bearing location can be calculated using the formula:

$$V = \frac{WS_v}{N} + \frac{M_{wsv}x_{beam}}{I_{beams}}$$

where N = number of beams

For example, at the bearing location for Beam 1 (left fascia beam with vertical load V<sub>1</sub>):

$$V_1 = \frac{-133.5}{6} + \frac{1713(22.5)}{1417.5} = 4.9 \text{ kips}$$

Table 11.4.3.9 summarizes the vertical wind on superstructure loads.

**Table 11.4.3.9 – Vertical Wind on Superstructure Loads (kips)**

Wind Attack Angle	V1	V2	V3	V4	V5	V6
0 Degree Skew	4.9	-5.9	-16.8	-27.7	-38.6	-49.4

**[3.8.1.2.3b]**

**Wind Load Applied Directly to the Substructure**

A wind load with the base wind pressures previously calculated for the Strength III and Strength V load combinations are resolved into components for different wind attack angles and applied directly to the pier. The resulting wind loads for Strength III are shown in Table 11.4.3.10. This wind load was applied as line loads to the pier cap and column members in the structural analysis model. Assuming 1 foot of cover over the tops of the footings, the projected area of the perpendicular face of the cap is 15.6 ft<sup>2</sup> and of the three columns is 49.7 ft<sup>2</sup>. The parallel face has an area of 229.4 ft<sup>2</sup> for the pier cap and 149.2 ft<sup>2</sup> for the columns.

**Table 11.4.3.10 – Wind Load on Substructure (Strength III)**

Wind Attack Angle	Pressure on Perpendicular Face (ksf)	Pressure on Parallel Face (ksf)
0 Degree Skew	0.054	0.000
15 Degree Skew	0.052	0.014
30 Degree Skew	0.047	0.027
45 Degree Skew	0.038	0.038
60 Degree Skew	0.027	0.047

**[3.8.1.3]**

**Wind on Live Load**

The wind on live load is the product of the base wind pressure and the tributary length which are the same for all applicable load combinations. The wind on live load was assumed to have the same tributary length as the superstructure wind load. It is applied at 6 feet above the top of the deck.

Tributary length for wind on live load:

$$L_{trib} = 2 \cdot 130 \cdot \frac{5}{8} = 162.5 \text{ ft}$$

The wind on live load WL is:

$$WL = P_B \cdot L_{\text{trib}}$$

where  $P_B$  = base wind pressure from LRFD Table 3.8.1.3-1 for various wind attack angles.

For example, for a wind attack angle skewed 30 degrees to the superstructure:

$$P_{\text{Btransv}} = P_{\text{Bpar}} = 0.082 \text{ klf}$$

$$P_{\text{Blong}} = P_{\text{Bperp}} = 0.024 \text{ klf}$$

$$WL_{\text{par}} = 0.082 \cdot 162.5 = 13.3 \text{ kips}$$

Then the lateral load L applied to the pier cap at each beam location is:

$$L_{\text{par1}} = L_{\text{par2}} = L_{\text{par3}} = L_{\text{par4}} = L_{\text{par5}} = L_{\text{par6}} = \frac{13.3}{6} = 2.2 \text{ kips}$$

$$WL_{\text{perp}} = 0.024 \cdot 162.5 = 3.9 \text{ kips}$$

Similar to the superstructure wind load, the longitudinal wind on live load component can be applied at the top of the pier for analysis. The height of application  $y_{\text{perp}}$  above the top of footing is:

$$y_{\text{perp}} = 26.75 - 4.50 = 22.25 \text{ ft}$$

Then for a wind attack angle skewed 30 degrees to the superstructure, the moment at the base of the columns is:

$$M_{\text{WLperp}} = 3.9 \cdot 22.25 \cdot \left(\frac{1}{3}\right) = 28.9 \text{ kip-ft/column}$$

Again, similar to the wind on superstructure load, the transverse wind on live load component will be applied at the centroid of the pier cap. This will require the addition of vertical loads at the bearings equivalent to the reduction in moment. For a wind attack angle skewed 30 degrees:

$$\begin{aligned} M_{\text{red}} &= WL_{\text{par}} \cdot (6 \text{ ft} + \text{dist. from top of deck to pier cap centroid}) \\ &= 13.3 \cdot \left(6.00 + (9.42 - 2.67) + \frac{4.67}{2}\right) = 200.6 \text{ kip-ft} \end{aligned}$$

Then for Beam 1 (left fascia beam with vertical load  $V_1$ ) and a wind attack angle skewed 30 degrees:

$$V_1 = \frac{M_{red} \cdot X_{beam}}{I_{beams}} = \frac{200.6 \cdot (22.5)}{1417.5} = 3.2 \text{ kips}$$

The wind on live load values are summarized in Table 11.4.3.11.

**Table 11.4.3.11 – Wind on Live Load**

Wind Attack Angle	L <sub>par1</sub> V <sub>1</sub> (kips)	L <sub>par2</sub> V <sub>2</sub> (kips)	L <sub>par3</sub> V <sub>3</sub> (kips)	L <sub>par4</sub> V <sub>4</sub> (kips)	L <sub>par5</sub> V <sub>5</sub> (kips)	L <sub>par6</sub> V <sub>6</sub> (kips)	M <sub>WLperp</sub> (kip-ft/col)
0 Degree Skew	2.7 3.9	2.7 2.3	2.7 0.8	2.7 -0.8	2.7 -2.3	2.7 -3.9	0.0
15 Degree Skew	2.4 3.4	2.4 2.1	2.4 0.7	2.4 -0.7	2.4 -2.1	2.4 -3.4	14.8
30 Degree Skew	2.2 3.2	2.2 1.9	2.2 0.6	2.2 -0.6	2.2 -1.9	2.2 -3.2	28.9
45 Degree Skew	1.8 2.6	1.8 1.5	1.8 0.5	1.8 -0.5	1.8 -1.5	1.8 -2.6	38.6
60 Degree Skew	0.9 1.3	0.9 0.8	0.9 0.3	0.9 -0.3	0.9 -0.8	0.9 -1.3	46.0

**C. Structural Analysis**

To determine the design forces in different portions of the pier a structural analysis was performed with a matrix analysis program. Gross section properties were used for all members. Fixed supports were provided at the top of each footing for column and cap design. Fixed supports were provided at the top of the piles for footing design.

The matrix analysis program was constructed using a beam-and-node model. The member representing the cap was set at the centroid of the cap. The longitudinal loads applied to the pier at the top of the cap were assumed to apply no moment to the pier due to bearing rotation and the force couple applied at the adjacent supports. However, this reduces the distance from the location of point loads to the support. A moment was added to account for this decrease in height. The value of the moment is equal to the force times the distance from the top of the cap to the centroid of the cap.

**D. Pier Cap Design**

**1. Design Loads**

The pier cap is designed for dead, live, and thermal loads. The braking load was assumed not to contribute to maximum vertical load effects for the design of the pier cap.

Wind loads can be neglected in the cap design. In load combinations with wind loading, the live load factor is reduced significantly. The additional vertical load from wind will not exceed the reduction in live load. Wind, wind on live load, and braking loads produce lateral loads on the pier cap that require calculation of Strength III and Strength V load combinations to determine the maximum moment and shear. In this example, these loads are very small and by inspection do not govern. For long spans between columns, deep caps, or thin caps, consider checking lateral loads to determine if skin and shear reinforcing are adequate for resistance.

Three load combinations are examined for design of the pier cap:

- Strength I is used to determine basic flexural and shear demands:

$$U_1 = 1.25 \cdot DC + 1.75 \cdot (LL + IM) + 0.50 \cdot TU$$

- Service I is used to check the distribution of flexural reinforcement (crack control):

$$S_1 = 1.00 \cdot DC + 1.00 \cdot (LL + IM) + 1.00 \cdot TU$$

- Fatigue I is used to ensure that adequate fatigue resistance is provided for an infinite life cycle:

$$F = 1.50 \cdot (LL + IM)$$

The pier cap design forces are listed in Tables 11.4.3.12 and 11.4.3.13.

For simplicity, negative bending moments are given at the column centerline. Another reasonable approach would be to use the average of the moments at the column centerline and the column face for the design negative moment.

Again for simplicity, pier cap shears at the columns are given at the column centerline. For pier configurations where beam reactions are located over a column, the design shear should be taken at the column face.

**Table 11.4.3.12 – Pier Cap Design Moments**

Load Combination	Positive Bending Moment (located at CL Beam 2) (kip-ft)	Negative Bending Moment (located at CL Column 1) (kip-ft)
Strength I	1618 (LL Case 4)	2356 (LL Case 3, left of CL)
Service I	1117 (LL Case 4)	1732 (LL Case 3, right of CL)
Fatigue I (max)	226 (LL Case 9)	251 (LL Case 11, left of CL)
Fatigue I (min)	-8 (LL Case 11)	0
Permanent Loads (unfactored)	470	1131 (left of CL) 1141 (right of CL)

**Table 11.4.3.13 – Pier Cap Design Shears (kips)**

Load Combination	Location Along Pier Cap				
	CL Beam 1	CL Column 1	CL Beam 2	CL Beam 3	CL Column 2
Strength I	619 (LL Case 3)	675 (LL Case 5)	660 (LL Case 5)	720 (LL Case 4)	733 (LL Case 4)
Permanent Loads (unfactored)	294	313	301	326	337

**2. Design Pier Cap Reinforcement for Bending Moment**

The flexural design of the cap is accomplished with four checks: flexural strength, crack control, fatigue, and minimum reinforcement. An appropriate level of reinforcement to satisfy the flexural force demand is computed first.

**[5.7.3.2]**

**Flexural Resistance**

Assume a rectangular stress distribution and solve for the required area of reinforcing based on  $M_u$  and  $d$ . For an  $f'_c$  of 4.0 ksi and a  $\beta_1$  of 0.85 the equation for the required area of steel reduces to:

$$M_u = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left[ d - \frac{a}{2} \right]$$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

$$M_u = \phi \cdot A_s \cdot f_y \cdot \left[ d - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b} \right]$$

$$M_u = 0.90 \cdot A_s \cdot (60) \cdot \left[ d - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 40} \right] \cdot \left[ \frac{1}{12} \right]$$

$$0.993 \cdot A_s^2 - 4.5 \cdot d \cdot A_s + M_u = 0$$

$$A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 3.972 \cdot M_u}}{1.986}$$

**[5.10.3.1.3]**

Compute “d” values for both a single layer of reinforcement (for positive moment) and a double layer of reinforcement (for negative moment). Assume the stirrups are #5 bars, the primary reinforcement is #9 bars, the clear cover is 2”, and the clear dimension between layers is 1.128” (which is the diameter of a #9 bar that can be used as a spacer between the 2 layers).

The “d” for a single layer of reinforcement is:

$$d = 56 - 2 - 0.625 - \frac{1.128}{2} = 52.81 \text{ in}$$

The assumed “d” for two layers of reinforcement is:

$$d = 56 - 2 - 0.625 - 1.128 - \frac{1.128}{2} = 51.68 \text{ in}$$

Using the Strength I design forces and assuming one layer of reinforcement for the positive moment steel and two layers of reinforcement for the negative steel, the required areas of steel can be found. They are presented in Table 11.4.3.14 along with trial reinforcement.

**Table 11.4.3.14 – Trial Longitudinal Reinforcement**

Location	M <sub>u</sub> (kip-ft)	d	A <sub>sreq</sub> (in <sup>2</sup> )	Trial Bars	A <sub>sprov</sub> (in <sup>2</sup> )
Positive Moment	1618	52.81	7.01	7-#9	7.00
Negative Moment	2356	51.68	10.57	14-#8	11.06

**[5.5.4.2.1-2]**

Verify the assumption that φ = 0.90.

For the positive moment reinforcement:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{7.00 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 40} = 3.63 \text{ in.}$$

**[5.7.2.1]**

Concrete compression strain limit ε<sub>c</sub> = 0.003

**[Table C5.7.2.1-1]**

Reinforcement tension-controlled strain limit ε<sub>tl</sub> = 0.005

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (52.81 - 3.63) \left( \frac{0.003}{3.63} \right) = 0.041 > \epsilon_{tl} = 0.005$$

Therefore, the initial assumption of  $\phi = 0.90$  is OK.

For the negative moment reinforcement:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{11.06 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 40} = 5.74 \text{ in.}$$

**[5.7.2.1]**

Since #8 bars were chosen for negative moment rather than the originally assumed #9, the depth "d" will have to be recalculated.

$$d = 56 - 2 - 0.625 - 1.00 - \frac{1.00}{2} = 51.88$$

Concrete compression strain limit  $\epsilon_c = 0.003$

**[Table C5.7.2.1-1]**

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (51.88 - 5.74) \left( \frac{0.003}{5.74} \right) = 0.024 > \epsilon_{tl} = 0.005$$

Therefore, the initial assumption of  $\phi = 0.90$  is OK.

**[5.7.3.4]**

**Positive Moment Crack Control**

To ensure that cracking is limited to small cracks that are well distributed, a limit is placed on the spacing and service load stress of the reinforcing steel.

The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. To simplify the calculations, the section is assumed to be singly reinforced.

**[5.4.2.4 & 5.7.1]**

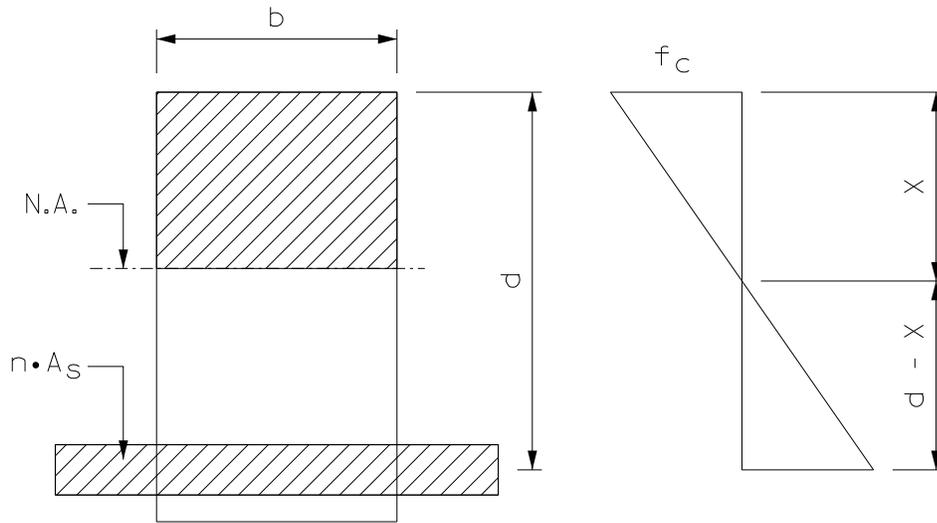
$$n = \frac{E_s}{E_c} = \frac{29,000}{33,000 \cdot (0.145)^{1.5} \cdot \sqrt{4.0}} = 7.96 \quad \text{Use } n=8$$

$$n \cdot A_s = 8 \cdot (7.00) = 56.00 \text{ in}^2$$

Referring to Figure 11.4.3.7, determine the location of the neutral axis:

$$b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)$$

$$\frac{(40) \cdot x^2}{2} = 56.00 \cdot (52.81 - x) \quad \text{solving, } x = 10.84 \text{ in}$$



**Figure 11.4.3.7  
Cracked Section Diagram**

Determine the lever arm between service load flexural force components.

$$j \cdot d = d - \frac{x}{3} = 52.81 - \frac{10.84}{3} = 49.20 \text{ in}$$

Compute the stress in the reinforcement.

$$\text{Actual } f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{1117 \cdot 12}{7.00 \cdot (49.20)} = 38.9 \text{ ksi}$$

Max allowable  $f_{ss} = 0.6 \cdot f_y = 36.0 \text{ ksi} < 38.9 \text{ ksi}$  NO GOOD

Increase the amount of steel by the ratio of the stresses:

$$A_s = \frac{38.9}{36.0} \cdot 7.00 = 7.56 \text{ in}^2$$

Try 8 - #9 bars,  $A_s = 8.00 \text{ in}^2$

Then:

$$\begin{aligned} n \cdot A_s &= 64.00 \text{ in}^2 \\ d &= 52.81 \text{ in} \\ x &= 11.50 \text{ in} \\ jd &= 48.98 \text{ in} \end{aligned}$$

Actual  $f_{ss} = 34.2 \text{ ksi} < 36.0 \text{ ksi}$  OK

Pier caps are designed using Class 2 exposure conditions, so  $\gamma_e = 0.75$ .

$$d_c = \text{clear cover} + \text{diameter of \#5 stirrup} + \frac{1}{2} \text{ diameter of \#9 bar} \\ = 2 + 0.625 + 1.128/2 = 3.19 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{3.19}{0.7 \cdot (56 - 3.19)} = 1.09$$

LRFD Equation 5.7.3.4-1 defines the maximum bar spacing permitted:

$$s_{\max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.09 \cdot 34.2} - 2 \cdot 3.19 = 7.70 \text{ in}$$

For 8 - #9 bars, actual bar spacing  $s$  is:

$$s = \frac{\text{pier cap width} - 2 \cdot \text{distance to center of bar}}{\text{Number of spaces}}$$

$$s = \frac{40 - 2 \cdot (2 + 0.625 + 0.5 \cdot 1.128)}{7} = 4.80 \text{ in} < 7.70 \text{ in} \quad \underline{\text{OK}}$$

### Positive Moment Fatigue

#### [5.5.3]

The stress range in the reinforcement is computed and compared against limits to ensure that adequate fatigue resistance is provided.

The unfactored dead load moment in the positive moment region is 470 kip-ft.

The extreme moments on the cross section when fatigue loading is applied are:

$$\text{Maximum moment} = 470 + 226 = 696 \text{ k-ft}$$

$$\text{Minimum moment} = 470 + (-8) = 462 \text{ k-ft}$$

Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:

For the maximum moment:

$$f_{\max} = \frac{M}{A_s \cdot j \cdot d} = \frac{696 \cdot 12}{8.00 \cdot (48.98)} = 21.3 \text{ ksi}$$

For the minimum moment:

$$f_{\min} = \frac{M}{A_s \cdot j \cdot d} = \frac{462 \cdot 12}{8.00 \cdot (48.98)} = 14.1 \text{ ksi}$$

The stress range in the reinforcement ( $f_r$ ) is the difference between the two stresses

$$f_r = (21.3 - 14.1) = 7.2 \text{ ksi}$$

**[5.5.3.2]**

The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.

$$f_{f(\max)} = 24 - \frac{20}{f_y} \cdot f_{\min} = 24 - \frac{20}{60} \cdot (14.1)$$

$$f_{f(\max)} = 19.3 \text{ ksi} > 7.2 \text{ ksi}$$

OK**Check Minimum Reinforcement for Positive Moment****[5.7.3.3.2]**

To prevent a brittle failure mode, adequate flexural reinforcement needs to be placed in the cross section. The modulus of rupture for normal weight concrete ( $\lambda = 1.0$ ) is:

**[5.4.2.6]**

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{4} = 0.48 \text{ ksi}$$

The gross moment of inertia is:

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 40 \cdot (56)^3 = 585,387 \text{ in}^4$$

The distance from the centroid to the tension face is:

$$y_t = 56/2 = 28.0 \text{ in}$$

Take  $\gamma_1 = 1.60$ ,  $\gamma_3 = 0.67$ , for ASTM A615 Grade 60 reinforcement.

Combining these terms to determine the cracking moment produces:

$$M_{cr} = \gamma_1 \cdot \gamma_3 \cdot \frac{f_r \cdot I_g}{y_t} = 1.6 \cdot 0.67 \cdot \frac{0.48 \cdot 585,387}{28.0 \cdot (12)} = 896.5 \text{ kip-ft}$$

The capacity of the steel provided is:

$$M_r = \phi A_s f_y (d - a/2)$$

$$= 0.9 \cdot (8.00) \cdot (60) \cdot \left[ 52.81 - \left( \frac{8.00 \cdot 60}{2 \cdot 0.85 \cdot 4 \cdot 40} \right) \right] \cdot \frac{1}{12}$$

$$= 1837.6 \text{ kip-ft} > 896.5 \text{ kip-ft}$$

OK

Provide 1 layer of 8 - #9 bars for positive reinforcement.

**[5.7.3.4]****Negative Moment Crack Control**

The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement.

For 2 layers of 7-#8 bars ( $A_s = 11.06 \text{ in}^2$ ) with a clear spacing between layers equal to 1.0 inch,

**[5.10.3.1.3]**

$$d = 56 - 2 - 0.625 - 1.00 - \frac{1.0}{2} = 51.88 \text{ in}$$

The transformed area of steel is:

$$n \cdot A_s = 8 \cdot (11.06) = 88.48 \text{ in}^2$$

The location of the neutral axis satisfies:

$$\frac{(40) \cdot x^2}{2} = 88.48 \cdot (51.88 - x) \quad \text{solving, } x = 13.1 \text{ in}$$

The lever arm between service load flexural force components is:

$$j \cdot d = d - \frac{x}{3} = 51.88 - \frac{13.1}{3} = 47.51 \text{ in}$$

And the stress in the reinforcement is:

$$\text{Actual } f_{ss} = \frac{M}{A_s \cdot j \cdot d} = \frac{1732 \cdot 12}{11.06 \cdot (47.51)} = 39.6 \text{ ksi}$$

Max allowable  $f_{ss} = 0.6 \cdot f_y = 36.0 \text{ ksi} < 39.6 \text{ ksi}$  NO GOOD

Increase the amount of steel by the ratio of the stresses:

$$A_s = \frac{39.6}{36.0} \cdot 11.06 = 12.17 \text{ in}^2$$

Try 2 layers of 8 - #8 bars,  $A_s = 12.64 \text{ in}^2$

Then:

$$n \cdot A_s = 101.12 \text{ in}^2$$

$$d = 51.88 \text{ in}$$

$$x = 13.86 \text{ in}$$

$$jd = 47.26 \text{ in}$$

Actual  $f_{ss} = 34.8 \text{ ksi} < 36.0 \text{ ksi}$  OK

For  $\gamma_e = 0.75$  and  $d_c = 3.13 \text{ in}$  (2" cover + 5/8" stirrup + 1/2 of #8 bar),

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{3.13}{0.7 \cdot (56 - 3.13)} = 1.08$$

The maximum spacing permitted of reinforcement is:

$$s_{\max} = \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = \frac{700 \cdot 0.75}{1.08 \cdot 34.8} - 2 \cdot 3.13 = 7.71 \text{ in}$$

For equal bar spacing,  $s = 4.82 \text{ in} < 7.71 \text{ in}$

OK

**[5.5.3]****Negative Moment Fatigue**

The moments on the negative moment section when fatigue loading is applied vary from:

$$\text{Maximum moment} = 1131 + 251 = 1382 \text{ k-ft}$$

$$\text{Minimum moment} = 1131 + 0 = 1131 \text{ k-ft}$$

Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:

For the maximum moment:

$$f_{\max} = \frac{M}{A_s \cdot j \cdot d} = \frac{1382 \cdot 12}{12.64 \cdot (47.26)} = 27.8 \text{ ksi}$$

For the minimum moment:

$$f_{\min} = \frac{M}{A_s \cdot j \cdot d} = \frac{1131 \cdot 12}{12.64 \cdot (47.26)} = 22.7 \text{ ksi}$$

The stress range in the reinforcement ( $f_r$ ) is the difference between the two stresses

$$f_r = (27.8 - 22.7) = 5.1 \text{ ksi}$$

**[5.5.3.2]**

The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.

$$f_{f(\max)} = 24 - \frac{20}{f_y} \cdot f_{\min} = 24 - \frac{20}{60} \cdot (22.7)$$

$$f_{f(\max)} = 16.4 > 5.1 \text{ ksi}$$

OK**[5.7.3.3.2]****Check Minimum Reinforcement for Negative Moment**

The compression block depth is:

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{12.64 \cdot 60}{0.85 \cdot 4 \cdot 40} = 5.58 \text{ in.}$$

The moment capacity provided is:

$$\begin{aligned} M_r &= \phi A_s f_y (d - a/2) \\ &= 0.9 \cdot (12.64) \cdot (60) \cdot \left[ 51.88 - \left( \frac{5.58}{2} \right) \right] \cdot \frac{1}{12} \\ &= 2792.2 \text{ kip-ft} > 896.5 \text{ kip-ft} \end{aligned}$$

OK

Provide 2 layers of 8-#8 bars ( $A_s = 12.64 \text{ in}^2$ ) for negative moment reinforcement.

### 3. Design Shear Reinforcement

The maximum factored design shear force is 733 kips (Strength I for Live Load Case 4) and occurs at the centerline of Column 2.

$$\text{Min. required } V_n = \frac{V_u}{\phi_v} = \frac{733}{0.90} = 814 \text{ kips}$$

The shear design for reinforced concrete elements is a two-step process. First, the shear capacity of the concrete section is determined. Second, the amount of shear steel is determined. The concrete capacity is dependent on  $\theta$ , the angle of inclination of the concrete struts, and  $\beta$ , a factor indicating the ability of the diagonally cracked concrete to transmit tension.

#### [5.8.3.4.1]

#### Determine Concrete Shear Capacity

The minimum shear reinforcement will be provided in the section.

Therefore,  $\beta = 2.0$  and  $\theta = 45$  degrees

$d_v$  is the distance between the internal flexural force components. The smaller distance between the "C" and "T" centroids is for the negative moment steel:

$$d_v = d - \frac{a}{2} = 51.88 - \frac{5.58}{2} = 49.1 \text{ in}$$

#### [5.8.2.9]

However,  $d_v$  need not be less than:

$$0.72 \cdot h = 0.72 \cdot 56 = 40.32 \text{ in}$$

or

$$0.90 \cdot d = 0.90 \cdot 51.88 = 46.69 \text{ in}$$

Use  $d_v = 49.1$  in

#### [5.4.2.8]

The concrete density modification factor,  $\lambda$ , for normal-weight concrete ( $W_c \geq 135$  pcf), is 1.0.

With  $d_v$  known, the concrete shear capacity can be computed:

#### [5.8.3.3-3]

$$V_c = 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.0316 \cdot 2 \cdot 1 \cdot \sqrt{4} \cdot 40 \cdot 49.1 = 248 \text{ kips}$$

#### Determine Stirrup Spacing

The difference between the required shear capacity and the capacity provided by the concrete is the required capacity for the shear steel.

$$\text{Min. required } V_s = (\text{Min. reqd. } V_n) - V_c = 814 - 248 = 566 \text{ kips}$$

Use #5 double "U" stirrups that will be vertical. Four legs of #5 bars have an area of:

$$A_v = 4 \cdot A_b = 4 \cdot 0.31 = 1.24 \text{ in}^2$$

**[5.8.3.3-4]**

The capacity of shear steel is:

$$V_s = \frac{[A_v \cdot f_y \cdot d_v \cdot \cot(\theta)]}{s}$$

This can be rearranged to solve for the stirrup spacing:

$$s \leq \frac{[A_v \cdot f_y \cdot d_v \cdot \cot(\theta)]}{V_s} = \frac{[1.24 \cdot 60 \cdot 49.1 \cdot \cot(45)]}{566} = 6.45 \text{ in}$$

To simplify construction, try a constant stirrup spacing of 6.0 inches between columns and in pier cap cantilever.

**[5.8.2.5]****Check Minimum Shear Reinforcement Requirements**

Determine maximum stirrup spacing that satisfies minimum transverse reinforcement requirements:

$$A_v \geq 0.0316 \cdot \lambda \cdot \sqrt{f'_c} \cdot \frac{b_v \cdot s}{f_y}$$

Rearranging and solving for stirrup spacing  $s$ ,

$$s \leq \frac{A_v \cdot f_y}{0.0316 \cdot \lambda \cdot b_v \cdot \sqrt{f'_c}} = \frac{1.24 \cdot 60}{0.0316 \cdot 1 \cdot 40 \cdot \sqrt{4}} = 29.4 \text{ in} \gg 6 \text{ in} \quad \underline{\text{OK}}$$

**[5.8.2.7]****Check Maximum Shear Reinforcement Spacing Requirements**

First determine if  $v_u < 0.125 \cdot f'_c$ :

**[5.8.2.9]**

$$0.125f'_c = 0.125 \cdot 4 = 0.50 \text{ ksi}$$

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{733 - 0}{0.9 \cdot 40 \cdot 49.1} = 0.41 \text{ ksi} < 0.50 \text{ ksi}$$

Therefore,

**[5.8.2.7]**

$$s_{\max} = 0.8d_v = 0.8 \cdot 49.1 = 39.3 \text{ in}$$

or

$$s_{\max} = 24.0 \text{ in}$$

$$s_{\max} = 24.0 \text{ in} \gg 6 \text{ in}$$

GOVERNS

OK

Use #5 double "U" stirrups at 6 inch spacing for shear reinforcement in the pier cap.

**[5.6.3]****4. Cantilever Capacity Check**

Check the capacity of the cantilever using the strut-and-tie method. A strut-and-tie model should be considered for the design of members where the distance between the center of applied load and the supporting reaction is less than twice the member thickness. Strut-and-tie models provide a way to approximate load patterns where conventional methods cannot due to a non-linear strain distribution. Begin by determining the vertical reaction applied to the cantilever.

Self weight of cantilever:

$$P_{\text{self}} = \left(\frac{40}{12}\right) \cdot \left(\frac{56 + 36}{2}\right) \cdot \left(\frac{1}{12}\right) \cdot 5.25 \cdot 0.150 = 10.1 \text{ kips}$$

Dead load from the superstructure is:

$$P_{\text{super}} = 287.3 \text{ kips}$$

The reaction from one lane of live load is:

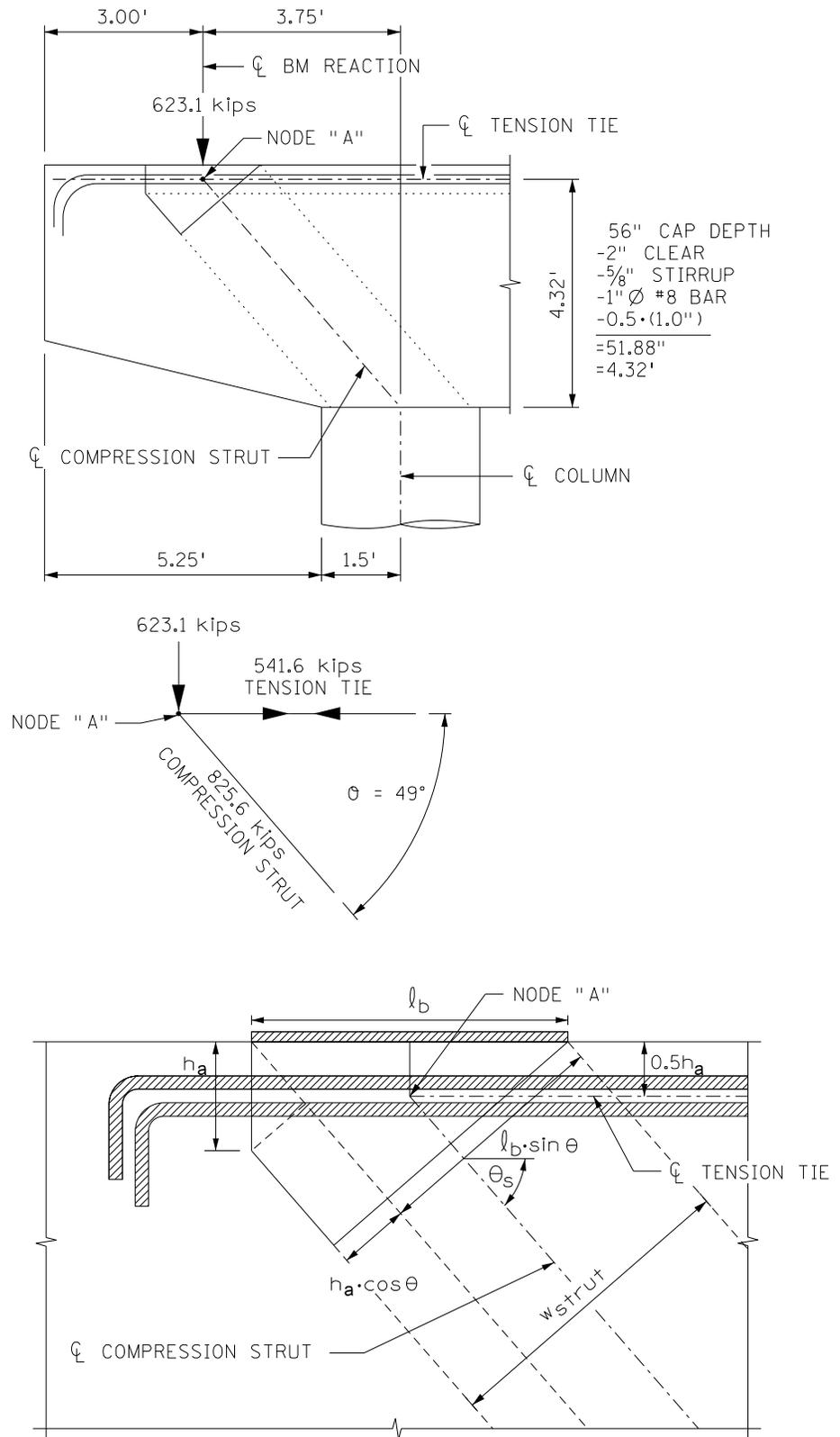
$$P_{\text{LL}} = 143.6 \text{ kips (LL Case 3)}$$

Then the factored vertical load on the cantilever is:

$$1.25 \cdot (10.1 + 287.3) + 1.75 \cdot (143.6) = 623.1 \text{ kips}$$

**[Fig. 5.6.3.3.2-1]**

Assume a simple model with a single horizontal tension tie centered on the top reinforcement and a single compression strut between the center of the tension tie below the bearing and the center of the column. For simplicity and conservatism, the pedestal was ignored. A schematic with the resultant loads in the strut and tie is shown in Figure 11.4.3.8.



**Figure 11.4.3.8**  
**Cantilever Strut and Tie Model**

**Tension Tie****[5.6.3.3]**

The required capacity of the tension tie  $P_{ntreq}$  is:

$$P_{ntreq} = \frac{T}{\phi} = \frac{541.6}{0.9} = 601.8 \text{ kips}$$

The tie is composed of 16-#8 bars ( $A_s = 12.64 \text{ in}^2$ ). The bars must be developed by the time they reach the inside face of the concrete strut.

**[5.11.2.4.1]**

The development length,  $\ell_{hb}$ , for deformed bars in tension terminating in a standard hook shall be greater than the smaller of:

- 8.0 bar diameters, and
- 6.0 in.

The concrete density modification factor,  $\lambda$ , for normal-weight concrete is 1.0.

**[5.4.2.8]**

The development length for tension tie #8 bars with standard hooks and  $\lambda = 1.0$ :

$$\ell_{hb} = \frac{38.0 \cdot d_b}{60.0} \cdot \frac{f_y}{\lambda \cdot \sqrt{f'_c}} = \frac{38.0 \cdot 1.00}{60.0} \cdot \frac{60}{1.0 \cdot \sqrt{4}} = 19.0 \text{ in}$$

**[5.11.2.4.2]**

The bars are epoxy coated with side cover  $\geq 2.5$  in, so  $\lambda_{cf} = 1.2$

The hook end cover  $\geq 2$  in, so  $\lambda_{rc} = 0.8$

Then  $\ell_{hb} = 19.0 \cdot 1.2 \cdot 0.8 = 18.2$  in

By inspection, the tension tie will be developed at the point where it intersects the inside face of the concrete strut.

The actual capacity of the tie is:

**[5.6.3.4.1]**

$$P_{nt} = f_y \cdot A_{st} = 60 \cdot 12.64 = 758.4 \text{ kips} > 601.3 \text{ kips} \quad \underline{\text{OK}}$$

**Strut-to-Node Interface Compression****[5.6.3.5.2]**

At node "A", the strut is anchored by bearing and reinforcement (CCT node). The effective cross-sectional area of the strut-to-node interface,  $A_{cnr}$ , is determined using the width and thickness of the strut. The width of the strut (measured along direction parallel to pier cap) is affected by the bearing pad width, the angle of the strut, and the height of the node back face. The bearing pads are 24 inches wide and the strut is inclined at 49 degrees from horizontal (Figure 11.4.3.8).

The height of the node back face  $h_a$  is:

$$h_a = 2 \cdot (2.0 + 0.625 + 1.0 + 1.0 \cdot 0.5) = 8.25 \text{ in}$$

Then the width of the strut  $W_{\text{strut}}$  is:

$$W_{\text{strut}} = \ell_b \cdot (\sin \theta) + h_a \cdot (\cos \theta)$$

$$= 24 \cdot (\sin 49) + 8.25 \cdot (\cos 49) = 23.53 \text{ in}$$

For the length of the strut (measured in direction perpendicular to pier cap), include the loaded length of the bearing pads plus the distance between the pads:

$$L_{\text{strut}} = 12 + 5 + 12 = 29.00 \text{ in}$$

Then the cross-sectional area of the strut-to-node interface is:

$$A_{\text{cn}} = W_{\text{strut}} \cdot L_{\text{strut}} = 23.53 \cdot 29.00 = 682.37 \text{ in}^2$$

### [5.6.3.5.3]

The limiting compressive stress at the node face is determined by the concrete compressive strength, confinement modification factor, and concrete efficiency factor.

Begin with the simple, conservative assumption that  $m = 1$ . If this does not work, we can refine by calculating  $A_1$  and  $A_2$  per AASHTO Article 5.6.3.5.3.

The concrete efficiency factor is dependent on whether crack control reinforcement per AASHTO Article 5.6.3.6 is provided. For this example, we will initially assume crack control reinforcement is not provided and use the reduced value for  $\nu = 0.45$ . If additional strength is needed, crack control bars will be added to increase the resistance.

The limiting compressive stress at the strut-to-node interface is:

$$f_{\text{cu}} = m \cdot \nu \cdot f'_c = 1.0 \cdot 0.45 \cdot 4.0 = 1.80 \text{ ksi} \quad \underline{\text{OK}}$$

### [5.6.3.3]

#### [5.6.3.5.1]

The factored resistance on the strut-to-node interface is :

$$\phi P_n = \phi \cdot A_{\text{cn}} \cdot f_{\text{cu}} = 0.7 \cdot 682.37 \cdot 1.80 = 859.8 \text{ kips} > 825.6 \text{ kips} \quad \underline{\text{OK}}$$

### Crack Control Reinforcement

Since adequate resistance is provided without the addition of crack control reinforcement, AASHTO Article 5.6.3.6 is waived.

### [5.7.3.4]

### 5. Longitudinal Skin Reinforcement

The effective depth for both positive and negative moment reinforcement is greater than 3.0 feet, so skin reinforcement is required. The minimum area of skin reinforcement required on each vertical face of the pier cap is:

Positive moment region:

$$A_{\text{sk}} \geq 0.012 \cdot (d_\ell - 30) = 0.012 \cdot (52.81 - 30) = 0.27 \text{ in}^2/\text{ft}$$

$$\text{but not more than } A_{sk} \leq \frac{A_s}{4} = \frac{8.00}{4} = 2.00 \text{ in}^2/\text{ft}$$

Negative moment region:

$$A_{sk} \geq 0.012(51.88 - 30) = 0.26 \text{ in}^2/\text{ft}$$

$$\text{but not more than } A_{sk} \leq \frac{12.64}{4} = 3.16 \text{ in}^2/\text{ft}$$

The skin reinforcement must be placed within  $d/2$  of the main reinforcement with a spacing not to exceed  $d/6$  or 12 inches.

Using the smallest  $d = 51.88$ ,

$$\frac{d}{2} = \frac{51.88}{2} = 25.94 \text{ in}$$

$$\frac{d}{6} = \frac{51.88}{6} = 8.65 \text{ in}$$

Choose 5-#5 bars equally spaced between the top and bottom reinforcement on each face. (Spacing = 7.86 in and  $A_s = 0.47 \text{ in}^2/\text{ft}$ )

### [5.10.8]

#### 6. Temperature Steel Check

A minimum amount of reinforcement needs to be provided to ensure that shrinkage and temperature cracks remain small and well distributed. The minimum amount required on each face and in each direction is:

$$\text{Total req'd } A_{s\text{req}} \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} = \frac{1.30 \cdot 40 \cdot 56}{2 \cdot (40 + 56) \cdot 60} = 0.25 \text{ in}^2/\text{ft}$$

$$\text{and } 0.11 \leq A_{s\text{req}} \leq 0.60$$

The actual total longitudinal reinforcement area on each vertical face is:

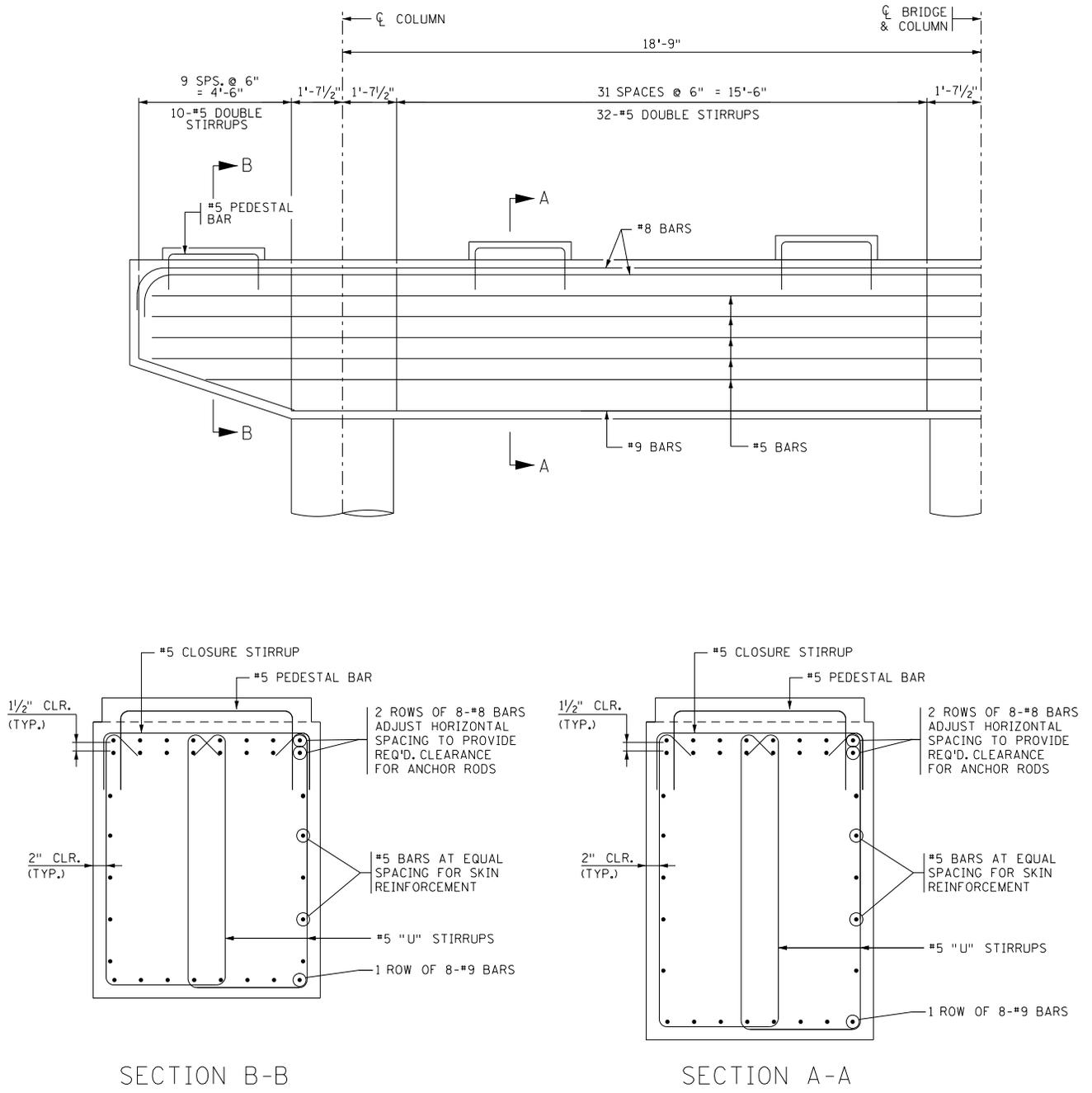
$$A_s = \frac{[2 \cdot (0.79) + 1 \cdot (1.00) + 5 \cdot (0.31)] \cdot 12}{56} = 0.89 \frac{\text{in}^2}{\text{ft}} > 0.25 \frac{\text{in}^2}{\text{ft}} \quad \underline{\text{OK}}$$

The actual total transverse reinforcement area on each face is:

$$A_s = \frac{0.31 \cdot 12}{6} = 0.62 \frac{\text{in}^2}{\text{ft}} > 0.25 \frac{\text{in}^2}{\text{ft}} \quad \underline{\text{OK}}$$

#### 7. Summary

Figure 11.4.3.9 details the final reinforcement in the pier cap.



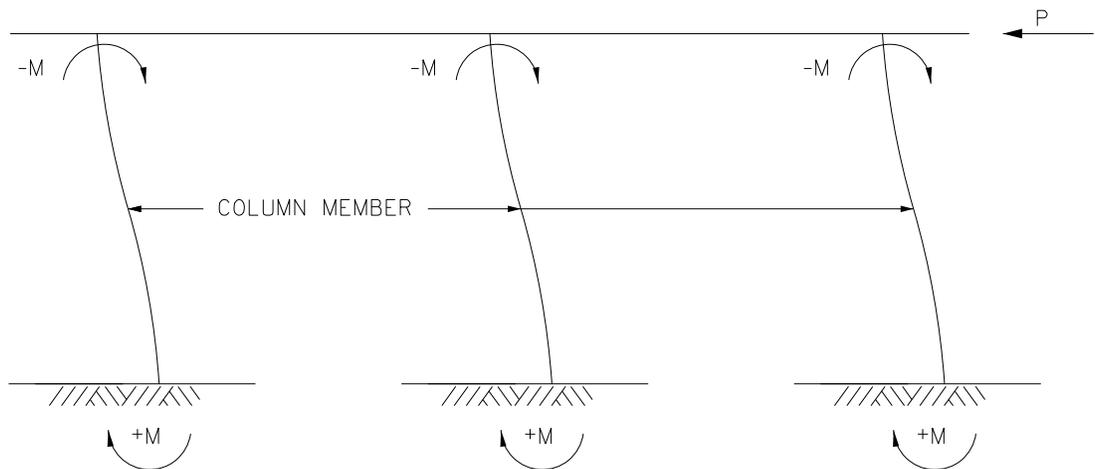
**Figure 11.4.3.9**  
**Pier Cap Reinforcement**

**E. Column Design**

**Design Forces**

Table 11.4.3.15 lists the unfactored axial loads and bending moments at the top and bottom of the columns when the pier is subjected to various loadings.

The sign convention for the axial loads is positive for downward forces and negative for upward forces. The sign convention for the bending moments in the parallel direction ( $M_{par}$ ) is beam convention. Positive moments cause tension on the "bottom side" of the column member which is defined as the right side of the column. Negative moments cause tension on the "top side" which is defined as the left side. (See Figure 11.4.3.10.)



**Figure 11.4.3.10**  
**Sign Convention for  $M_{par}$**

For moments in the perpendicular direction ( $M_{perp}$ ), all lateral loads are assumed applied in the same direction. Therefore, all moments are shown as positive.

Moments shown in the table due to wind transverse to the bridge are based on a wind directed from right to left. (Column 3 is on the windward side of the pier.)

**Table 11.4.3.15 – Unfactored Column Member Forces (k, k-ft)**

Load	Force	Column 1 (Leeward)		Column 2 (Center)		Column 3 (Windward)	
		Top	Bottom	Top	Bottom	Top	Bottom
Dead Load	P	616	637	674	695	616	637
	M <sub>par</sub>	-10	5	0	0	10	-5
	M <sub>perp</sub>	0	0	0	0	0	0
Live Load Case 1 One Lane	P	108	108	137	137	-10	-10
	M <sub>par</sub>	-72	32	59	-32	7	-6
	M <sub>perp</sub>	0	0	0	0	0	0
Live Load Cases 2 and 3 One Lane (max/min)	P	234/19	234/19	196/9	196/9	19/-5	19/-5
	M <sub>par</sub>	47/-40	19/-12	0/-6	13/0	40/-20	20/-19
	M <sub>perp</sub>	0	0	0	0	0	0
Live Load Cases 4, 5, and 6 Two Lanes (max/min)	P	248/47	248/47	297/156	297/156	47/-13	47/-13
	M <sub>par</sub>	-46/-82	38/25	67/0	0/-34	66/-4	5/-32
	M <sub>perp</sub>	0	0	0	0	0	0
Live Load Case 7 Three Lanes	P	97	97	304	304	97	97
	M <sub>par</sub>	-79	38	0	0	79	-38
	M <sub>perp</sub>	0	0	0	0	0	0
Live Load Case 8 Four Lanes	P	146	146	215	215	146	146
	M <sub>par</sub>	-34	16	0	0	34	-16
	M <sub>perp</sub>	0	0	0	0	0	0
Braking	P	0	0	0	0	0	0
	M <sub>par</sub>	0	0	0	0	0	0
	M <sub>perp</sub>	28	267	28	268	28	267
45 °F Temperature Drop	P	-9	-9	18	18	-9	-9
	M <sub>par</sub>	129	-141	0	0	-129	141
	M <sub>perp</sub>	0	0	0	0	0	0
35° F Temperature Rise	P	7	7	-14	-14	7	7
	M <sub>par</sub>	-100	109	0	0	100	-109
	M <sub>perp</sub>	0	0	0	0	0	0
Wind at 0° on Superstructure and Substructure (Strength III)	P	30	30	0	0	-30	-30
	M <sub>par</sub>	-214	235	-239	247	-214	235
	M <sub>perp</sub>	0	0	0	0	0	0
Wind at 15° on Superstructure and Substructure (Strength III)	P	27	27	0	0	-27	-27
	M <sub>par</sub>	-191	210	-214	221	-191	210
	M <sub>perp</sub>	6	86	6	86	6	86
Wind at 30° on Superstructure and Substructure (Strength III)	P	25	25	0	0	-25	-25
	M <sub>par</sub>	-176	193	-197	203	-176	193
	M <sub>perp</sub>	13	179	13	180	13	179
Wind at 45° on Superstructure and Substructure (Strength III)	P	20	20	0	0	-20	-20
	M <sub>par</sub>	-142	155	-158	164	-142	155
	M <sub>perp</sub>	17	236	16	237	17	236
Wind at 60° on Superstructure and Substructure (Strength III)	P	10	10	0	0	-10	-10
	M <sub>par</sub>	-73	81	-82	85	-73	81
	M <sub>perp</sub>	21	288	19	290	21	288
Wind at 0° on Superstructure and Substructure (Strength V)	P	15	15	0	0	-15	-15
	M <sub>par</sub>	-103	113	-115	119	-103	113
	M <sub>perp</sub>	0	0	0	0	0	0
Wind at 15° on Superstructure and Substructure (Strength V)	P	13	13	0	0	-13	-13
	M <sub>par</sub>	-88	97	-99	102	-88	97
	M <sub>perp</sub>	4	48	3	48	4	48

**Table 11.4.3.15 – Unfactored Column Member Forces (k, k-ft) (cont’d)**

Load	Force	Column 1 (Leeward)		Column 2 (Center)		Column 3 (Windward)	
		Top	Bottom	Top	Bottom	Top	Bottom
Wind at 30° on Superstructure and Substructure (Strength V)	P	12	12	0	0	-12	-12
	M <sub>par</sub>	-82	90	-92	95	-82	90
	M <sub>perp</sub>	6	83	6	84	6	83
Wind at 45° on Superstructure and Substructure (Strength V)	P	10	10	0	0	-10	-10
	M <sub>par</sub>	-69	76	-77	80	-69	76
	M <sub>perp</sub>	8	116	8	117	8	116
Wind at 60° on Superstructure and Substructure (Strength V)	P	5	5	0	0	-5	-5
	M <sub>par</sub>	-35	38	-39	40	-35	38
	M <sub>perp</sub>	10	137	9	138	10	137
Vertical Wind	P	2	2	-46	-46	-89	-89
	M <sub>par</sub>	4	2	0	4	4	2
	M <sub>perp</sub>	0	0	0	0	0	0
Wind on Live Load at 0°	P	11	11	0	0	-11	-11
	M <sub>par</sub>	-50	55	56	58	-50	55
	M <sub>perp</sub>	0	0	0	0	0	0
Wind on Live Load at 15°	P	9	9	0	0	-9	-9
	M <sub>par</sub>	-44	48	50	51	-44	48
	M <sub>perp</sub>	2	15	-2	15	2	15
Wind on Live Load at 30°	P	9	9	0	0	-9	-9
	M <sub>par</sub>	-41	44	46	47	-41	44
	M <sub>perp</sub>	3	29	-3	29	3	29
Wind on Live Load at 45°	P	7	7	0	0	-7	-7
	M <sub>par</sub>	-33	36	38	38	-33	36
	M <sub>perp</sub>	4	39	-4	39	4	39
Wind on Live Load at 60°	P	4	4	0	0	-4	-4
	M <sub>par</sub>	-17	18	19	19	-17	18
	M <sub>perp</sub>	5	46	-5	46	5	46

The following three limit states are examined for the columns:

$$\text{Strength I: } U_1 = \gamma_p \cdot DC + 1.75 \cdot LL + 1.75 \cdot BR + 0.50 \cdot TU$$

$$\text{Strength III: } U_3 = \gamma_p \cdot DC + 1.00 \cdot WS + 0.50 \cdot TU$$

Strength V:

$$U_5 = \gamma_p \cdot DC + 1.35 \cdot LL + 1.35 \cdot BR + 1.00 \cdot WS + 1.00 \cdot WL + 0.50 \cdot TU$$

Load combinations were tabulated for the appropriate limit states for each of the various live load cases, wind angles, the temperature rise and fall, and also for maximum and minimum DC load factors.

Then the worst case loadings (maximum axial load with maximum moment, maximum moment with minimum axial load) were chosen from each limit state from the tabulated load combinations. These are shown

in Table 11.4.3.16. The critical cases for the column among those listed in the table are shown in bold print.

**Table 11.4.3.16 - Column Design Forces**

Load Combination		Axial Load P (kips)	M <sub>par</sub> (kip-ft)	M <sub>perp</sub> (kip-ft)
Strength I:	(a) Column 1 Bottom: $\gamma_D = 1.25$ , LL Case 5, BR, $\Delta$ Temp = +35°F	1234	105	467
	<b>(b) Column 2 Bottom: <math>\gamma_D = 1.25</math>, LL Case 7, BR, <math>\Delta</math> Temp = -45°F</b>	<b>1410</b>	<b>0</b>	<b>469</b>
	(c) Column 3 Bottom: $\gamma_D = 1.25$ , LL Case 8, BR, $\Delta$ Temp = +35°F	1055	89	467
	(d) Column 3 Bottom: $\gamma_D = 0.90$ , LL Case 5, BR, $\Delta$ Temp = -45°F	546	75	467
Strength III:	(a) Column 2 Bottom: $\gamma_D = 1.25$ , Wind Skew = 60°, $\Delta$ Temp = -45°F	878	119	406
	(b) Column 3 Bottom: $\gamma_D = 0.90$ , Wind Skew = 60°, $\Delta$ Temp = -45° F	555	179	403
	<b>(c) Column 3 Bottom: <math>\gamma_D = 0.90</math>, Wind Skew = 0°, Vertical Wind, <math>\Delta</math> Temp = -45° F</b>	<b>402</b>	<b>398</b>	<b>0</b>
Strength V:	<b>(a) Column 1 Bottom: <math>\gamma_D = 0.90</math>, LL Case 6, BR, Wind Skew = 60°, <math>\Delta</math> Temp = +35°F</b>	<b>646</b>	<b>135</b>	<b>461</b>
	(b) Column 2 Bottom: $\gamma_D = 1.25$ , LL Case 7, BR, Wind Skew = 60°, $\Delta$ Temp = -45°F	1288	35	463
	<b>(c) Column 3 Bottom: <math>\gamma_D = 0.90</math>, LL Case 3, BR, Wind Skew = 60°, <math>\Delta</math> Temp = -45°F</b>	<b>556</b>	<b>126</b>	<b>461</b>

**[5.7.4.3]**

**Slenderness Effects**

Each column is considered unbraced in both the parallel and perpendicular directions. The dimension "L" from bottom of pier cap to top of footing is 17.58 feet.

In the parallel direction, a fixed condition exists at the bottom and a rotation-fixed, translation-free condition exists at the top. For this condition LRFD Table C4.6.2.5-1 recommends a K value of 1.20.

Then:

$r$  = radius of gyration of a circular column

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\pi \cdot d^4}{\frac{\pi \cdot d^2}{4}}} = d/4$$

$$r = 0.25 \cdot (\text{column diameter}) \\ = 0.25 \cdot (3) = 0.75 \text{ ft}$$

$$\left(\frac{KL}{r}\right)_{\text{par}} = \frac{1.2(17.58)}{0.75} = 28.1 > 22$$

Therefore, slenderness effects need to be considered for the parallel direction.

In the perpendicular direction the columns can conservatively be considered as cantilevers fixed at the bottom. For this condition LRFD Table C4.6.2.5-1 recommends a  $K$  value of 2.1.

Then:

$$\left(\frac{KL}{r}\right)_{\text{perp}} = \frac{2.1(17.58)}{0.75} = 49.2 > 22$$

Therefore, slenderness effects need to be considered for the perpendicular direction, also.

Two choices are available to designers when including slenderness effects in the design of columns. A moment magnification method is described in LRFD Article 4.5.3.2.2. The other method is to use an iterative  $P-\Delta$  analysis.

A  $P-\Delta$  analysis was used for this example. For simplicity and in order to better match the computer model used, take the column height  $L$  equal to the distance from the top of footing to the centroid of the pier cap. Calculations are shown below for the Strength  $V(c)$  load case.

For the perpendicular direction, the factored moment and corresponding axial load from Table 11.4.3.16 is:

$$M_{\text{perp}} = 461 \text{ kip-ft}, \quad P = 646 \text{ kips} \quad (\text{Strength } V(c))$$

Then the maximum equivalent lateral force  $H_{\text{perp}}$  applied at the top of the column is:

$$H_{\text{perp}} = \frac{M_{\text{perp}}}{L} = \frac{461}{19.92} = 23.1 \text{ kips}$$

This force produces a perpendicular displacement  $\Delta_{\text{perp}}$  at the top of the column:

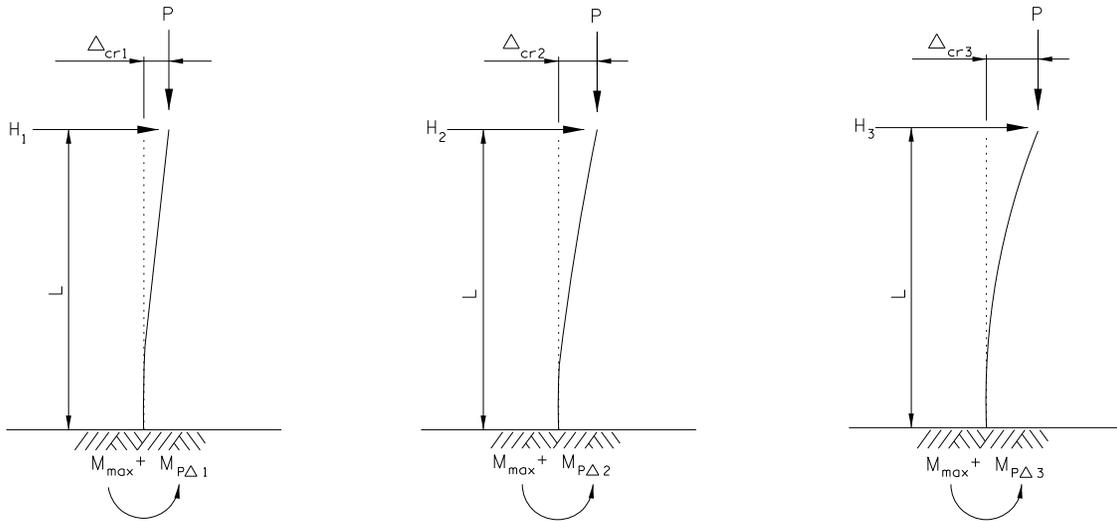
$$\Delta_{\text{perp}} = \frac{H_{\text{perp}} L^3}{3EI} = \frac{23.1 \cdot [(19.92)(12)]^3}{3 \cdot (3644) \cdot (82448)} = 0.350 \text{ in}$$

The structural model used in the analysis contained gross section properties. To account for the reduced stiffness of a cracked column section, the displacement was multiplied by an assumed cracked section factor equal to 2.5. This factor is based on using LRFD Equation 5.7.4.3-2 with  $\beta_d$  equal to zero and corresponds to 40 percent of the gross section properties being effective. (Other references suggest values ranging from 30 percent to 70 percent be used for columns.) After updating the equivalent lateral force for the P- $\Delta$  moment, three additional iterations were performed. The final longitudinal displacement was found to be 0.389 inches and the additional perpendicular moment due to slenderness was 52.4 kip-feet. See Figure 11.4.3.12 and Table 11.4.3.17 for a summary of the perpendicular direction P- $\Delta$  analysis for the Strength V(c) limit state.

For the parallel direction, the corresponding factored moment from Table 11.4.3.16 is:

$$M_{\text{par}} = 135 \text{ kip} - \text{ft} \quad (\text{Strength V (c)})$$

A procedure similar to that done for the perpendicular direction was used for the P- $\Delta$  analysis. For the parallel direction, equations used to compute  $H_{\text{par}}$  and  $\Delta_{\text{par}}$  are for a cantilever column fixed at one end and free to deflect horizontally but not rotate at the other end (taken from *Manual of Steel Construction, LRFD Design, Thirteenth Edition*, page 3-218). For this example, values of  $\Delta H$  converged after 2 iterations. In practice, more iterations may be required. See Figure 11.4.3.13 and Table 11.4.3.18 for a summary of the parallel direction P- $\Delta$  analysis. This process was repeated for the other three critical load cases shown in Table 11.4.3.16.



$$H_1 = \frac{M_{\max}}{L}$$

$$\Delta_{g1} = \frac{H_1 L^3}{3EI}$$

$$\Delta_{cr1} = F_{cr} \cdot \Delta_{g1}$$

$$M_{P\Delta 1} = P \cdot \Delta_{cr1}$$

$$\Delta H_1 = \frac{M_{P\Delta 1}}{L}$$

$$H_2 = H_1 + \Delta H_1$$

$$\Delta_{g2} = \frac{H_2 L^3}{3EI}$$

$$\Delta_{cr2} = F_{cr} \cdot \Delta_{g2}$$

$$M_{P\Delta 2} = P \cdot \Delta_{cr2}$$

$$\Delta H_2 = \frac{M_{P\Delta 2}}{L}$$

$$H_3 = H_1 + \Delta H_2$$

$$\Delta_{g3} = \frac{H_3 L^3}{3EI}$$

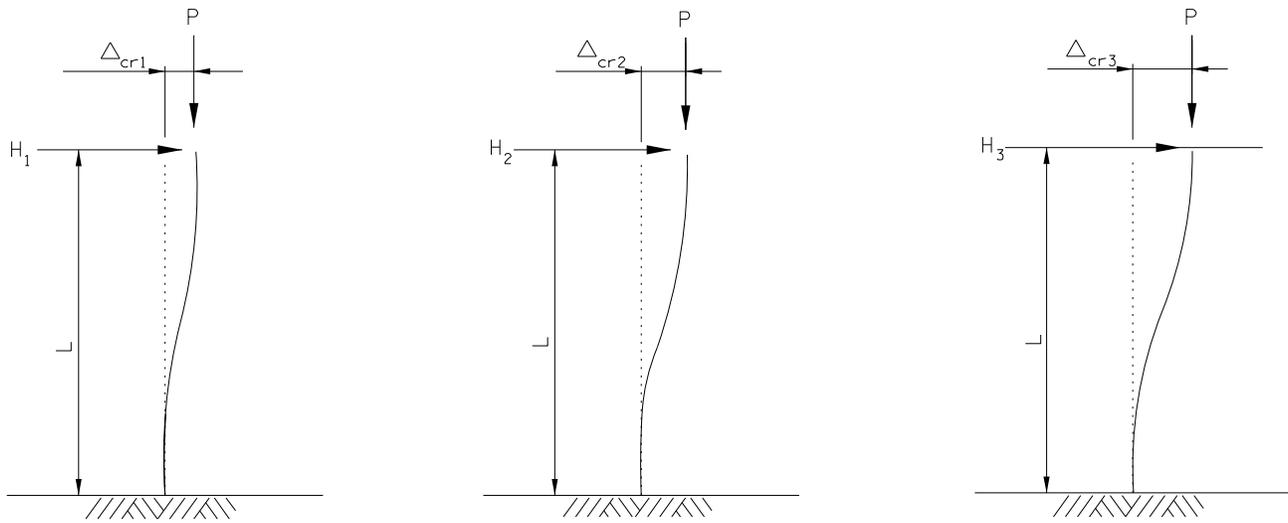
$$\Delta_{cr3} = F_{cr} \cdot \Delta_{g3}$$

$$M_{P\Delta 3} = P \cdot \Delta_{cr3}$$

**Figure 11.4.3.12**  
**Perpendicular Direction P-Δ Procedure**

**Table 11.4.3.17 – Perpendicular P-Δ Moment**

Equiv. Lateral Force H (kips/column)	Axial Load P (kips/column)	Δ <sub>g</sub> for gross section properties (in)	Cracked Section Factor F <sub>cr</sub>	Δ <sub>cr</sub> for cracked section (in)	M <sub>PΔ</sub> (k-ft)	ΔH to produce M <sub>PΔ</sub> (kips)
23.1	646	0.350	2.5	0.875	47.1	2.4
25.5	646	0.386	2.5	0.965	51.9	2.6
25.7	646	0.389	2.5	0.973	52.4	2.6
Add 52 k-ft to column for slenderness in the perpendicular direction						



$$H_1 = \frac{2M_{max}}{L}$$

$$\Delta_{g1} = \frac{H_1 L^3}{12EI_g}$$

$$\Delta_{cr1} = F_{cr} \cdot \Delta_{g1}$$

$$M_{P\Delta 1} = P \cdot \Delta_{cr1}$$

$$\Delta H_1 = \frac{2 \cdot M_{P\Delta 1}}{L}$$

$$H_2 = H_1 + \Delta H_1$$

$$\Delta_{g2} = \frac{H_2 L^3}{12EI}$$

$$\Delta_{cr2} = F_{cr} \cdot \Delta_{g2}$$

$$M_{P\Delta 2} = P \cdot \Delta_{cr2}$$

$$\Delta H_2 = \frac{2 \cdot M_{P\Delta 2}}{L}$$

$$H_3 = H_1 + \Delta H_2$$

$$\Delta_{g3} = \frac{H_3 L^3}{12EI}$$

$$\Delta_{cr3} = F_{cr} \cdot \Delta_{g3}$$

$$M_{P\Delta 3} = P \cdot \Delta_{cr3}$$

**Figure 11.4.3.13**  
**Parallel Direction P-Δ Procedure**

**Table 11.4.3.18 – Parallel P-Δ Moment**

Equiv. Lateral Force H (kips/column)	Axial Load P (kips/column)	Δ <sub>g</sub> for gross section properties (in)	Cracked Section Factor F <sub>cr</sub>	Δ <sub>cr</sub> cracked section (in)	M <sub>PΔ</sub> (k-ft)	ΔH to produce M <sub>PΔ</sub> (kips)
13.55	646	0.0513	2.5	0.1283	6.9	0.69
14.24	646	0.0539	2.5	0.1348	7.3	0.73
14.28	646	0.0541	2.5	0.1353	7.3	0.73

Add 7 k-ft to column for slenderness in the parallel direction

The design forces presented in Table 11.4.3.19 are the factored axial loads and resultant moments that include P-Δ effects. Because of the symmetry of the round cross section, the moments in the parallel and perpendicular directions can be combined using the square root of the sum of the squares (Pythagorean Theorem).

$$M_R = \sqrt{M_{par}^2 + M_{perp}^2}$$

**Table 11.4.3.19 – Critical Column Design Forces (kips, kip-ft)**

Load Combination	Axial Load	M <sub>par</sub>	M <sub>par</sub> P-Δ	Total M <sub>par</sub>	M <sub>perp</sub>	M <sub>perp</sub> P-Δ	Total M <sub>perp</sub>	Resultant M <sub>R</sub>
Strength I (a)	1410	0	0	0	469	135	604	604
Strength III (c)	402	398	13	411	0	0	0	411
Strength V (a)	646	135	7	142	461	52	513	532
Strength V (c)	556	126	6	132	461	44	505	522

The minimum amount of column reinforcement must be such that:

[5.7.4.2-3]

$$\frac{A_s f_y}{A_g f'_c} \geq 0.135$$

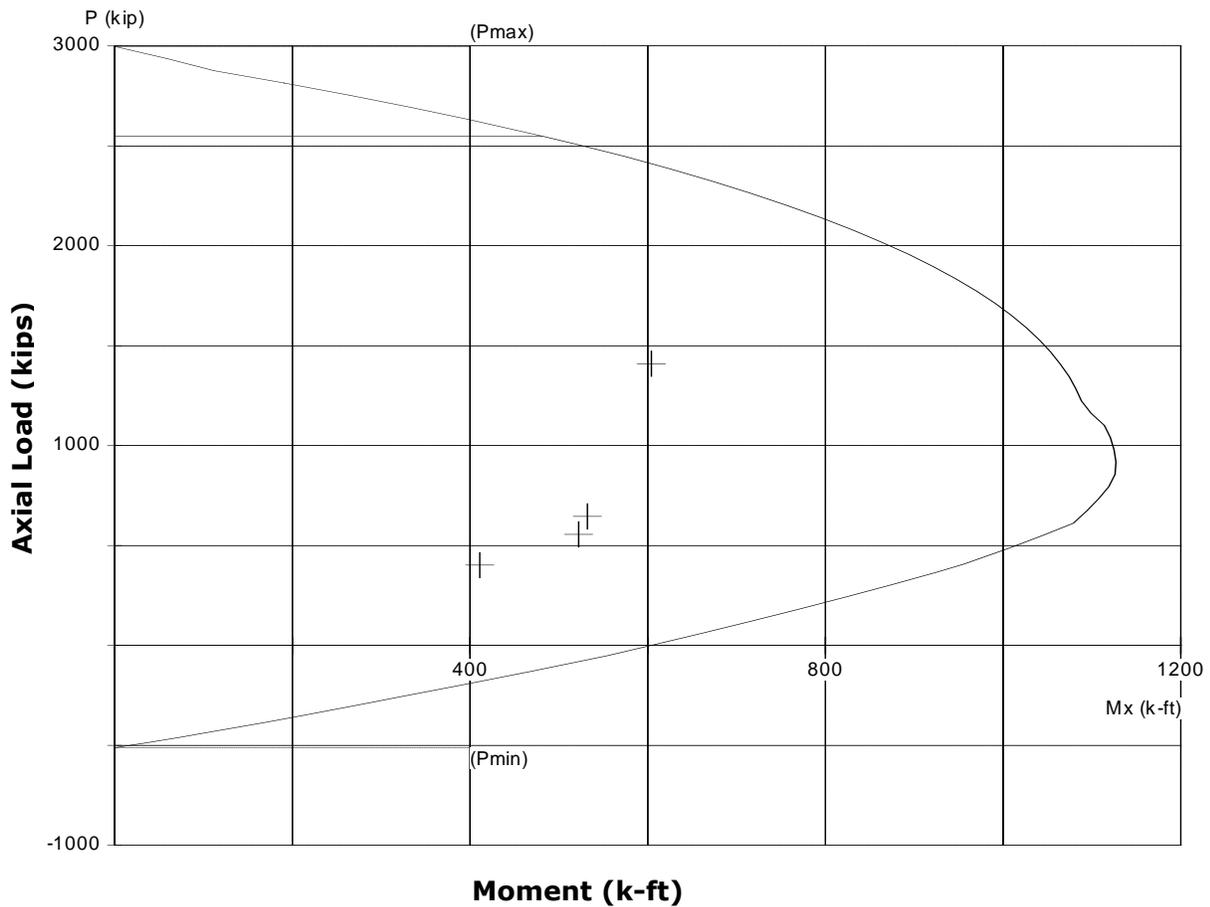
Then:

$$\text{Min } A_s \geq \left( \frac{A_g f'_c}{f_y} \right) \cdot 0.135 = \left( \frac{1018 \cdot 4.0}{60.0} \right) \cdot 0.135 = 9.16 \text{ in}^2$$

Try 12-#8 bars (A<sub>s</sub>= 9.48 in<sup>2</sup>).

A computer program was used to generate the column strength interaction diagram shown in Figure 11.4.3.13. The figure also displays the design axial loads and moments for the critical load cases. All values fall well within the capacity of the column.

The interaction diagram includes φ factors of 0.90 for flexure and 0.75 for axial compression.



**Figure 11.4.3.13**  
**Column Interaction Curve For**  
**36 inch Diameter Column With 12-#8 Bars**

**[5.7.4.2]**

**Reinforcement Limit Check**

For non-prestressed columns the maximum amount of longitudinal reinforcement permitted is:

$$\frac{A_s}{A_g} = \frac{9.48}{1018} = 0.00931 \leq 0.08 \quad \text{OK}$$

**[5.10.6.2]**

**Column Spirals**

Per MnDOT standard practice, use spiral reinforcing for columns with diameters up to 42". Use #4E bars with a 3" pitch for the spiral. The anchorage of the spiral reinforcement shall be provided by 1½ extra turns of spiral bar at each end of the spiral unit.

**[5.7.4.6]**

Check reinforcement ratio of spiral to concrete core:

$$\rho_s = \frac{\text{volume of spiral in one loop}}{\text{volume of core for one pitch spacing}}$$

For a clear cover of 2", diameter of the core  $D_c = 32$  in.

Spiral reinforcement area  $A_{sp} = 0.20$  in<sup>2</sup>

Spiral bar diameter  $d_b = 0.50$  in

Pitch spacing  $p = 3.0$  in

Length of one loop

$$\ell_{sp} = \sqrt{[\pi \cdot (D_c - d_b)]^2 + p^2} = \sqrt{[\pi \cdot (32 - 0.50)]^2 + (3.0)^2} = 99.01 \text{ in}$$

Then actual

$$\rho_s = \frac{A_{sp} \cdot \ell_{sp}}{\left(\frac{\pi \cdot D_c^2}{4}\right) \cdot p} = \frac{0.20 \cdot 99.01}{\left(\frac{\pi \cdot 32^2}{4}\right) \cdot 3.0} = 0.00821$$

Required minimum

$$\begin{aligned} \rho_s &= 0.45 \cdot \left(\frac{A_g}{A_c} - 1\right) \cdot \frac{f'_c}{f_{yh}} \\ &= 0.45 \cdot \left[\frac{1018}{\left(\frac{\pi \cdot 32^2}{4}\right)} - 1\right] \cdot \frac{4}{60} = 0.00797 < 0.00821 \quad \text{OK} \end{aligned}$$

**F. Piling Design****Loads**

A different computer model was used for the piling and footing design than used previously for the cap and column design. The column in the revised model extends from the centroid of the cap to the top of the piling (1'-0" above the footing bottom). The braking, wind, and temperature loads applied to the revised model remain the same as those applied in the cap and column design. Additional loads included for the piling and footing design include the weight of the footing and an assumed 1'-0" of earth.

Additional DC due to footing:

$$P = 10.0 \cdot 13.0 \cdot 4.50 \cdot 0.150 = 87.8 \text{ kips}$$

$$M_{par} = M_{perp} = 0 \text{ kip-ft}$$

Earth above footing EV:

$$P = 1.0 \cdot \left( (10 \cdot 13) - \frac{1018}{144} \right) \cdot (0.120) = 14.8 \text{ kips}$$

$$M_{\text{par}} = M_{\text{perp}} = 0 \text{ kip} - \text{ft}$$

For the earth loads, use a maximum load factor of 1.35 and a minimum load factor of 0.90.

Also, the dynamic load allowance is removed from the live load when designing foundation components entirely below ground.

The procedure for computing the critical loads for piling design is the same as for determining the loads at the bottom of the column. However, for the piling design, the focus is on load combinations that maximize the axial load and the bending moment. Also, since the piling layout is not identical in both the perpendicular and parallel direction, it is possible that a load combination different than what was critical for the columns could govern the piling design.

The values for the maximum loadings for piling design are shown in Table 11.4.3.20.

**Table 11.4.3.20 – Piling Design Forces**

Load Combination		Axial Load (kips)	M <sub>par</sub> (kip-ft)	M <sub>par</sub> P-Δ (kip-ft)	Total M <sub>par</sub> Bending Moment (kip-ft)	M <sub>perp</sub> (kip-ft)	M <sub>perp</sub> P-Δ (kip-ft)	Total M <sub>perp</sub> Bending Moment (kip-ft)
Str I	(a) Column 1: γ <sub>D</sub> =1.25, LL Case 5, Δ Temp = +35°F	1298	81	12	93	540	186	726
	(b) Column 2: γ <sub>D</sub> =1.25, LL Case 7, Δ Temp = -45°F	1455	0	0	0	542	221	763
	(c) Column 2: γ <sub>D</sub> =1.25, LL Case 4, Δ Temp = -45°F	1319	44	7	51	542	190	732
Str III	(a) Column 1: γ <sub>D</sub> =1.25, LL Case 7, Wind skew=60°, Δ Temp = -45°F	946	179	18	197	464	101	565
	(b) Column 2: γ <sub>D</sub> =1.25, LL Case 7, Wind skew=15°, Δ Temp = -45°F	1005	139	15	154	466	111	577
Str V	(a) Column 1: γ <sub>D</sub> =1.25, LL Case 5, Wind skew=60°, Δ Temp = +35°F	1220	112	15	127	533	167	700
	(b) Column 2: γ <sub>D</sub> =1.25, LL Case 7, Wind skew=60°, Δ Temp = -45°F	1352	41	6	47	534	195	729

**Determine Required Number of Piles**

As a starting point, estimate the number of piles needed by calculating the number of piles required to resist the largest axial load and then add 10 to 20% more piles to resist overturning.

$$N_{axial} = \left( \frac{1455}{200} \right) = 7.3 \text{ piles}$$

Try the trial pile layout presented in Figure 11.4.3.15 with 10 piles.

Knowing the loads applied to the footing and the layout of the piles, the force in each pile can be determined. The equation to be used is:

$$P = \left[ \frac{\text{Axial Load}}{\text{Number of Piles}} \right] + \left[ \frac{M_{\text{par}} \cdot X_{\text{par}}}{\sum X_{\text{par}}^2} \right] + \left[ \frac{M_{\text{perp}} \cdot X_{\text{perp}}}{\sum X_{\text{perp}}^2} \right]$$

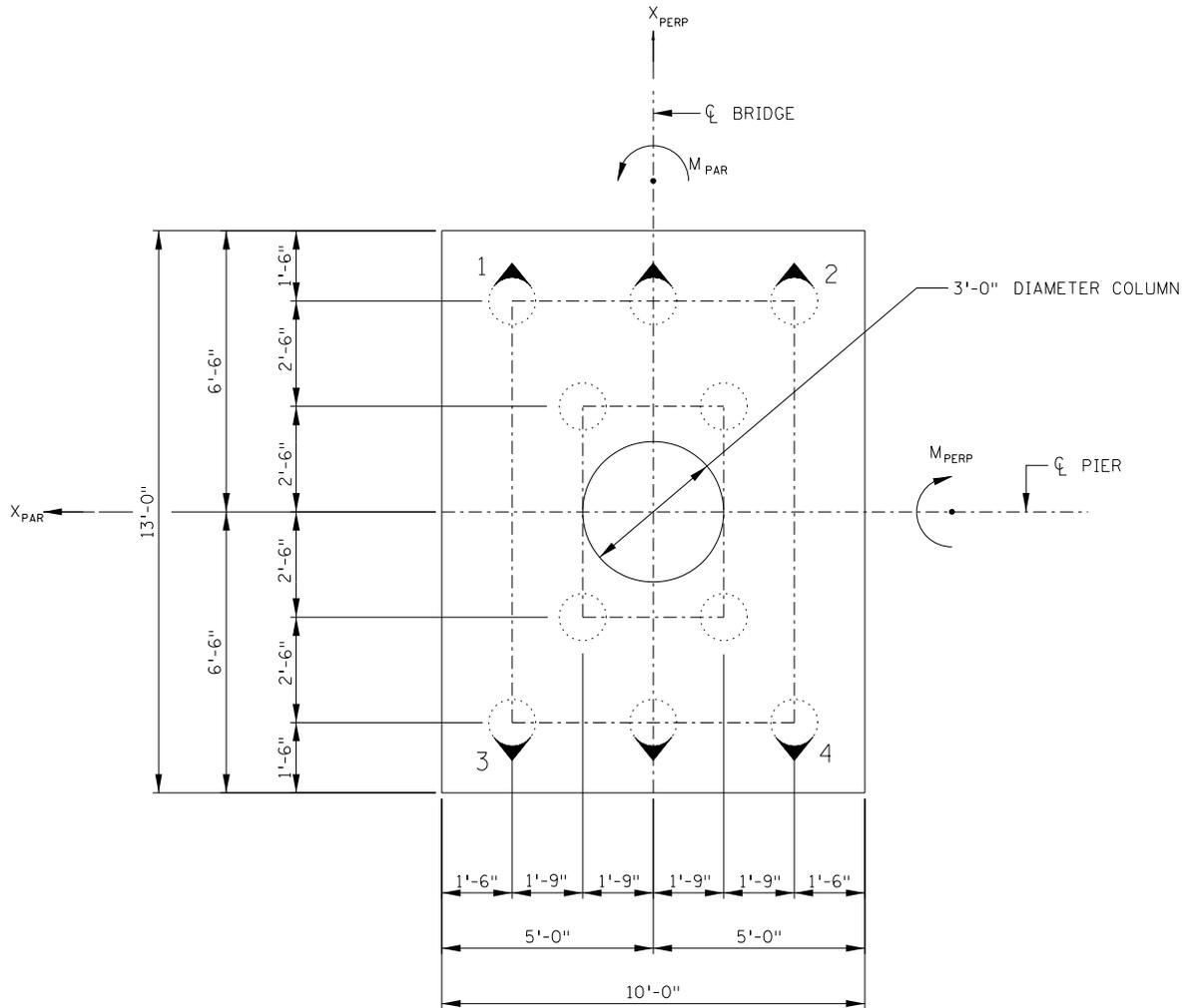
The equation assumes that the footing functions as a rigid plate and that the axial force in the piles due to applied moments is proportional to the distance from the center of the pile group.

$$\sum X_{\text{par}}^2 = 2 \cdot 3.50^2 + 2 \cdot 1.75^2 + 2 \cdot 0^2 + 2 \cdot (-1.75)^2 + 2 \cdot (-3.50)^2 = 61.25 \text{ ft}^2$$

$$\sum X_{\text{perp}}^2 = 3 \cdot 5.00^2 + 2 \cdot 2.50^2 + 1 \cdot 0^2 + 2 \cdot (-2.50)^2 + 3 \cdot (-5.00)^2 = 175.00 \text{ ft}^2$$

Then, for example, the Strength I(a) Corner Pile 1 load is:

$$P = \frac{1298}{10} + \frac{93 \cdot 3.50}{61.25} + \frac{726 \cdot 5.00}{175.00} = 155.9 \text{ kips}$$



**Figure 11.4.3.15**  
**Trial Pile Layout**

The factored pile loads at each corner of the footing (as identified in Figure 11.4.3.15) are presented in Table 11.4.3.21. All are below the 200 kip capacity of the piles.

**Table 11.4.3.21 – Factored Pile Loads**

Load Combination	Corner Pile Loads (kips)			
	1	2	3	4
Strength I(a)	155.9	145.2	114.4	103.7
Strength I(b)	167.3	167.3	123.7	123.7
Strength I(c)	155.7	149.9	113.9	108.1
Strength III(a)	122.0	99.5	89.7	67.2
Strength III(b)	125.8	108.2	92.8	75.2
Strength V(a)	149.3	134.7	109.3	94.7
Strength V(b)	158.7	153.3	117.1	111.7

### Pile Load Tables for Plan

Piles are driven until dynamic equation measurements indicate the pile has reached refusal or the required design load indicated in the plan. The nominal pile bearing resistance is monitored in the field using the MnDOT Pile Formula 2012 (MPF12) given in Article 2452.3.E.3 of the *MnDOT Standard Specifications For Construction, 2016 Edition*. Designers must calculate the pile load for the critical load case and show it in the plan, using the Standard Plan Note tables for piers with piling (see Appendix 2-H of this manual).

The critical load case for the pier piling is:

Strength I at Column 2 with  $\gamma = 1.25$ ,  
Live Load Case 7, and  $\Delta \text{Temp.} = -45^\circ \text{ F}$ .

The separated unfactored forces are:

$$P_{DL} = 783 \text{ kips}, M_{DLpar} = M_{DLperp} = 0 \text{ kip-ft}$$

$$P_{EV} = 14.8 \text{ kips}, M_{EVpar} = M_{EVperp} = 0 \text{ kip-ft}$$

$$P_{LL} = 257 \text{ kips}, M_{LLpar} = M_{LLperp} = 0 \text{ kip-ft (w/o dyn. load allowance)}$$

$$P_{BR} = 0 \text{ kips}, M_{BRpar} = 0 \text{ kip-ft}, M_{BRperp} = 309 \text{ kip-ft}$$

$$P_{TU} = 13 \text{ kips}, M_{TUpar} = M_{TUperp} = 0 \text{ kip-ft}$$

$$M_{P\Delta par} = 0 \text{ kip-ft}, M_{P\Delta perp} = 221 \text{ kip-ft (note that } P\Delta \text{ effects are based on factored loads)}$$

First, compute separate factored pile loads due to dead load, live load, and overturning load for load table:

$$\text{Factored } P_{DL} \text{ (includes EV)} = 1.25 \cdot 783 + 1.35 \cdot 14.8 = 998.7 \text{ kips}$$

$$\text{Factored } M_{DL,par} = \text{Factored } M_{DL,perp} = 0 \text{ kip-ft}$$

$$\text{Factored Pile Dead Load} = \left( \frac{998.7}{10} \right) \cdot \frac{1}{2} = 49.9 \text{ tons/pile}$$

$$\text{Factored } P_{LL} \text{ (w/o dynamic load allowance)} = 1.75 \cdot 257 = 449.8 \text{ kips}$$

$$\text{Factored } M_{LL,par} = \text{Factored } M_{LL,perp} = 0 \text{ kip-ft}$$

$$\text{Factored Pile Live Load} = \left( \frac{449.8}{10} \right) \cdot \frac{1}{2} = 22.5 \text{ tons/pile}$$

$$\text{Factored } P_{OT} = 0.50 \cdot 13 = 6.5 \text{ kips}$$

$$\text{Factored } M_{OT,par} = 0 \text{ kip-ft}$$

$$\text{Factored } M_{OT,perp} = 1.75 \cdot (309) + 221 = 761.8 \text{ kip-ft}$$

$$\text{Factored Pile OT Load} = \left( \frac{6.5}{10} + \frac{0 \cdot 3.50}{61.25} + \frac{761.8 \cdot 5.00}{175.00} \right) \cdot \frac{1}{2} = 11.2 \text{ tons/pile}$$

Factored Design Pile Load = 49.9 + 22.5 + 11.2 = 83.6 tons/pile

The final results to be shown in the plan are:

PIER COMPUTED PILE LOAD – TONS/PILE	
FACTORED DEAD LOAD	49.9
FACTORED LIVE LOAD	22.5
FACTORED OVERTURNING	11.2
* FACTORED DESIGN LOAD	83.6

\* BASED ON STRENGTH I LOAD COMBINATION

PIER REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES R <sub>n</sub> – Tons/Pile		
FIELD CONTROL METHOD	φ <sub>dyn</sub>	** R <sub>n</sub>
MNDOT PILE FORMULA 2012 (MPF12)  $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	167.2
PDA	0.65	128.6

\*\*R<sub>n</sub> = (FACTORED DESIGN LOAD) / φ<sub>dyn</sub>

**G. Footing Design**

**Check Shear Capacity of Footing**

Using a column diameter of 3'-0" and a footing thickness of 4'-6", the critical sections for shear and flexure for the footing can be found. Begin by determining the width of an equivalent square column.

$$A = \frac{\pi \cdot D^2}{4} = 1018 = b^2 \qquad b = 31.9 \text{ in, say } 32 \text{ in}$$

The critical section for one-way shear is located a distance d<sub>v</sub> away from the face of the equivalent square column. Two-way shear is evaluated on a perimeter located d<sub>v</sub>/2 away from the face of the actual round column. The same dimension d<sub>v</sub>/2 is used to check two-way shear for a corner pile.

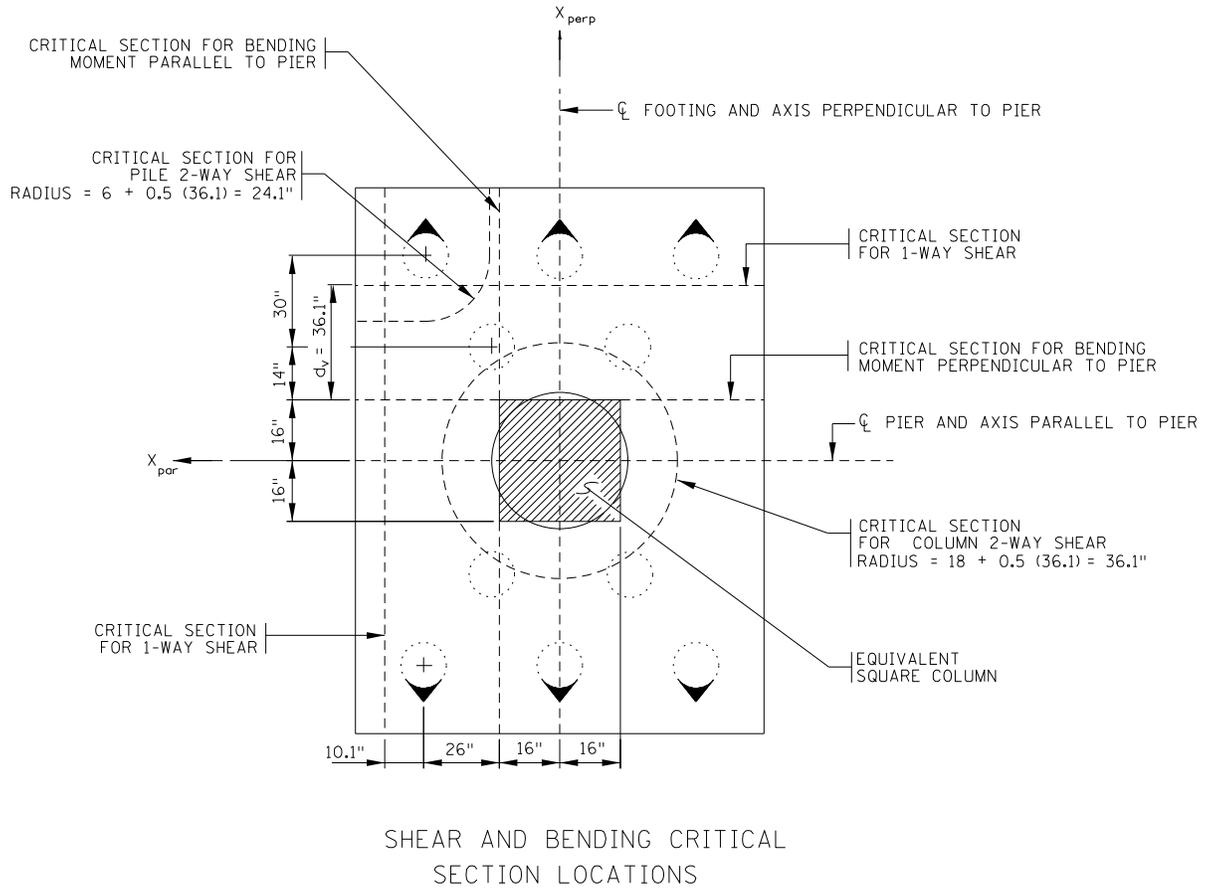
**[5.8.2.9]**

Estimate d<sub>v</sub> as 0.9d<sub>e</sub>. Note that it is not appropriate to use 0.72h here because the tension reinforcement is located so high above the bottom of the footing.

Conservatively calculate d<sub>e</sub> by assuming #10 bars in both directions and that the bars sit directly on top of the piles. Use the inside bar for d<sub>e</sub> calculation. If shear capacity is a problem, check that the d<sub>e</sub> value being

used corresponds to the critical section under investigation. If it doesn't, revise  $d_e$  and  $d_v$  values and recalculate shear capacity.

Then estimated  $d_v = 0.9d_e = 0.9 \cdot (54 - 12 - 1.27 - \frac{1}{2} \cdot 1.27) = 36.1$  in



**Figure 11.4.3.16**

The critical section for flexure is located at the face of the equivalent square column. All of the critical sections are presented in Figure 11.4.3.16.

**Check One-Way Shear**

The critical one-way shear section is located 36.1 inches away from the face of the equivalent square column.

For the portion of the footing that extends parallel to the pier all of the piles are within the critical shear section and no check is necessary.

For the portion of the footing that extends perpendicular to the pier, the three outermost piles lie outside of the critical shear section and the sum reaction must be resisted.

$$V_u = 167.3 \cdot 3 = 501.9 \text{ kips}$$

**[5.8.3.3]**

The one-way shear capacity of the footing is:

$$\phi V_c = \phi \cdot 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

**[5.8.3.4.1]**

The point of zero shear must be within  $3 \cdot d_v$  of the column face to be able to assume  $\beta = 2.0$ . Since  $3 \cdot d_v = 108.3''$ , by inspection, the point of zero shear is within acceptable parameters. Therefore, it can be assumed that  $\beta = 2.0$ .

**[5.4.2.8]**

For normal weight concrete,  $\lambda = 1.0$ .

Then:

$$\begin{aligned} \phi V_c &= 0.90 \cdot 0.0316 \cdot 2 \cdot 1.0 \cdot \sqrt{4} \cdot (10 \cdot 12) \cdot 36.1 \\ &= 492.8 \text{ kips} < 501.9 \text{ kips} \quad 1.8\% \text{ under, say } \underline{OK} \end{aligned}$$

**[5.13.3.6.3]****Check punching shear around the column**

Assume the entire column vertical load needs to be carried at the perimeter. If the footing has inadequate capacity, reduce the demand by subtracting piles and dead load "inside" of the perimeter.

The perimeter for two-way shear is:

$$b_o = 2 \cdot \pi \cdot 36.1 = 226.8 \text{ in}$$

Punching shear capacity is:

$$\phi V_n = \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \lambda \cdot \sqrt{f'_c} \cdot b_o \cdot d_v \leq 0.126 \cdot \lambda \cdot \sqrt{f'_c} \cdot b_o \cdot d_v$$

The aspect ratio of the column ( $\beta_c$ ) is 1.0. By inspection, the upper limit will govern.

$$\begin{aligned} \phi V_n &= \phi V_c = \phi \cdot 0.126 \cdot \lambda \cdot \sqrt{f'_c} \cdot b_o \cdot d_v = 0.90 \cdot 0.126 \cdot 1.0 \cdot \sqrt{4} \cdot 226.8 \cdot 36.1 \\ &= 1857 \text{ kips} > 1455 \text{ kips} \quad \underline{OK} \end{aligned}$$

**Check punching shear on a corner pile**

The critical shear section is assumed to be  $0.5d_v$  away from the outside edge of the pile. The shear section path with the shortest distance to the edge of the footing will provide the smallest capacity.

$$b_o = \frac{2 \cdot \pi \cdot 24.1}{4} + 18 + 18 = 73.9 \text{ in}$$

Once again using  $\beta_c$  equal to 1.0, inserting values into LRFD Equation 5.13.3.6.3-1 produces:

$$\begin{aligned}\phi V_n = \phi V_c &= \phi \cdot 0.126 \cdot \lambda \cdot \sqrt{f'_c} \cdot b_o \cdot d_v = 0.90 \cdot 0.126 \cdot 1.0 \cdot \sqrt{4} \cdot 73.9 \cdot 36.1 \\ &= 605.1 \text{ kips} > 167.3 \text{ kips} \qquad \text{OK}\end{aligned}$$

[5.7.2.2]

[5.7.3.2]

### Design Footing Reinforcement Perpendicular to Pier for Factored Moments

Determine the required area of flexural reinforcement to satisfy the Strength I(b) Load Combination. Five piles contribute to the design moment at the critical section for moment perpendicular to the pier. The three outer piles are located 44" away from the critical section. The two inner piles are located 14" away from the critical section.

$$P_{\text{inner}} = \frac{1455}{10} + \frac{763 \cdot 2.50}{175.00} = 156.4 \text{ kips/pile}$$

Then the design moment on the critical section is:

$$M_u = \left( 3 \cdot 167.3 \cdot \frac{44}{12} \right) + \left( 2 \cdot 156.4 \cdot \frac{14}{12} \right) = 2205 \text{ kip-ft}$$

Set up the equation to solve for the required area of steel assuming that  $\phi = 0.90$ :

$$\begin{aligned}M_u &= \phi \cdot A_s \cdot f_y \cdot \left[ d - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b} \right] \\ M_u &= 0.90 \cdot A_s \cdot (60) \cdot \left[ d - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 120} \right] \cdot \left[ \frac{1}{12} \right]\end{aligned}$$

$$\begin{aligned}0.3309 \cdot A_s^2 - 4.5 \cdot d \cdot A_s + M_u &= 0 \\ A_s &= \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 1.3236 \cdot M_u}}{0.6618}\end{aligned}$$

To compute "d" use the previous assumption that #10 bars are used for both mats of reinforcement and that they rest directly on top of the cut off piles. In addition, reduce "d" to permit either set of bars to rest directly on the pile.

$$d = \left[ 54 - 12 - 1.27 - \frac{1.27}{2} \right] = 40.10 \text{ in}$$

The required area of steel is 12.51 in<sup>2</sup>. Try 10-#10 bars spaced at 12 inches. The provided area of steel is 12.70 in<sup>2</sup>.

Confirm the initial assumption that  $\phi=0.90$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{12.70 \cdot 60}{0.85 \cdot 4 \cdot 120} = 1.87 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{1.87}{0.85} = 2.20 \text{ in.}$$

**[5.7.2.1]**

Concrete compression strain limit  $\epsilon_c = 0.003$

**[Table C5.7.2.1-1]**

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (40.10 - 2.20) \left( \frac{0.003}{2.20} \right) = 0.0517 > \epsilon_{tl} = 0.005$$

Therefore, the initial assumption of  $\phi = 0.90$  is OK.

**[5.7.3.4]**

#### **Crack Control**

Crack control checks are not performed on footings.

**[5.5.3]**

#### **Fatigue**

By inspection, fatigue is not checked for footings.

**[5.7.3.3.2]**

#### **Check Minimum Reinforcement**

The modulus of rupture is:

**[5.4.2.6]**

$$f_r = 0.24 \cdot \lambda \cdot \sqrt{f'_c} = 0.24 \cdot 1.0 \cdot \sqrt{4} = 0.48 \text{ ksi}$$

The gross moment of inertia is:

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 120 \cdot (54)^3 = 1,574,640 \text{ in}^4$$

The distance from the tension face to the centroid is:

$$y_t = 27.0 \text{ in}$$

Using  $\gamma_1 = 1.6$  and  $\gamma_3 = 0.67$  for ASTM 615 Grade 60 reinforcement,

$$M_{cr} = \gamma_3 \cdot \gamma_1 \cdot \frac{f_r \cdot I_g}{y_t} = 0.67 \cdot 1.6 \cdot \frac{0.48 \cdot 1,574,640}{27.0 \cdot (12)} = 2501 \text{ kip-ft}$$

The capacity of the section must be  $\geq$  the smaller of:

$$M_{cr} = 2501 \text{ kip-ft}$$

GOVERNS

$$\text{or } 1.33M_u = 1.33 \cdot 2205 = 2933 \text{ kip-ft}$$

The resisting moment is:

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9 (12.70) \cdot (60) \cdot \left[ 40.10 - \frac{1.87}{2} \right] \cdot \frac{1}{12}$$

$$= 2238 \text{ kip-ft} < 2501 \text{ kip-ft} \quad \text{NO GOOD}$$

Revise reinforcement to 12-#10 bars spaced at 10 inches ( $A_s = 15.24 \text{ in}^2$ ) with standard hooks.

$$M_r = 2673 \text{ kip-ft} > 2501 \text{ kip-ft} \quad \text{OK}$$

[5.7.2.2]  
[5.7.3.2]

**3. Design Footing Reinforcement Parallel to Pier For Factored Moments**

Determine the required area of flexural reinforcement to satisfy the Strength I load combination for parallel moments. Four piles contribute to the design moment at the critical section for moment parallel to the pier.

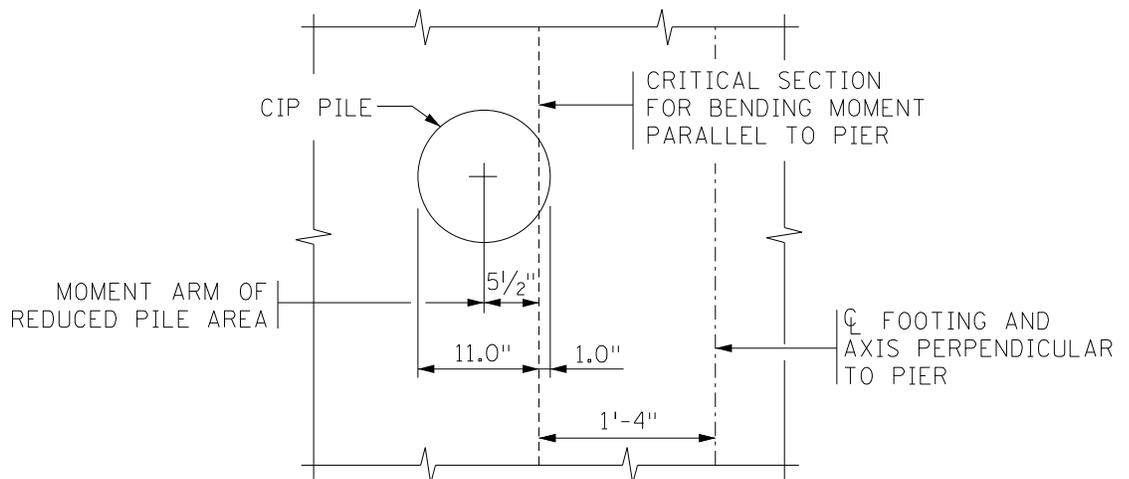
Piles 1 and 3 have reaction of 167.3 kips and 123.7 kips respectively. The inner pile above the  $x_{par}$  axis was previously shown to have a reaction equal to 156.4 kips.

The pile reaction for the inner pile below the  $X_{par}$  axis is:

$$P = \frac{1455}{10} + \frac{763 \cdot (-2.50)}{175.00} = 134.6 \text{ kips}$$

[5.13.3.6.1]

The inner piles lie partially inside of the critical section. Only the portion of the reaction outside the critical section causes moment at the critical section. See Figure 11.4.3.17.



**Figure 11.4.3.17**  
**Partial Footing Plan**

Then the design moment on the critical section is:

$$M_u = (167.3 + 123.7) \cdot \left(\frac{26}{12}\right) + (156.4 + 134.6) \cdot \left(\frac{11}{12}\right) \cdot \left(\frac{5.5}{12}\right) = 753 \text{ kip-ft}$$

Using the same "d" value of 40.10 inches as used for the perpendicular reinforcement, the required area of steel is 4.26 in<sup>2</sup>. Try 13-#6 bars spaced at 12 inches. The provided area of steel is 5.72 in<sup>2</sup>.

**[5.5.4.2.1]**

Again, confirm the initial assumption that  $\phi=0.90$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{5.72 \cdot 60}{0.85 \cdot 4.0 \cdot 156} = 0.65 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{0.65}{0.85} = 0.76 \text{ in}$$

$$d_t = 54 - 12 - 1.27 - 0.5 \cdot 0.75 = 40.36$$

Concrete compression strain limit  $\epsilon_c = 0.003$

**[5.7.2.1]**

**[Table C5.7.2.1-1]**

Reinforcement tension-controlled strain limit  $\epsilon_{tl} = 0.005$

$$\epsilon_t = (d - c) \left( \frac{\epsilon_c}{c} \right) = (40.36 - 0.76) \left( \frac{0.003}{0.76} \right) = 0.156 > \epsilon_{tl} = 0.005$$

Therefore, the initial assumption of  $\phi = 0.90$  is OK.

**[5.7.3.4]**

**Crack Control**

Crack control checks are not performed on footings.

**[5.5.3]**

**Fatigue**

By inspection, fatigue is not checked for footings.

**[5.7.3.3.2]**

**Check Minimum Reinforcement**

Revise the  $M_{cr}$  value computed earlier for a footing length of 13 feet:

**[5.4.2.6]**

$$M_{cr} = 2501 \cdot \frac{13}{10} = 3251 \text{ kip-ft}$$

The minimum required flexural resistance is the lesser of  $M_{cr}$  or:

$$1.33M_u = 1.33 \cdot 753 = 1001 \text{ kip-ft}$$

GOVERNS

The resisting moment is:

$$M_r = \phi A_s f_y (d - a/2) = 0.9 (5.72) \cdot (60) \cdot \left[ 40.36 - \frac{0.65}{2} \right] \cdot \frac{1}{12}$$

$$= 1031 \text{ kip-ft} > 1001 \text{ kip-ft} \quad \text{OK}$$

Provide 13-#6 bars spaced at 12 inches ( $A_s = 5.72 \text{ in}^2$ ) with standard hooks.

#### [5.11.2.1.1]

#### 4. Dowel Bar Development and Lap Splice

Determine the lap length for the primary column steel to dowel splice.

All primary column steel bars are spliced at the same location, consequently the lap is a Class B splice. The primary column reinforcement consists of #8 epoxy coated bars. For ease of construction, the dowel circle will be detailed to the inside of the column bar circle. Accordingly, the dowels will be increased one size to #9 bars.

#### [5.11.2.1.1]

For the #9 dowels:

$$\text{Clear cover } d_{\text{dowclr}} = 2.00 + 0.50 + 1.00 = 3.50 \text{ in}$$

$$\text{Dowel circle diameter} = \pi \cdot [d_{\text{col}} - 2 \cdot (d_{\text{dowclr}} + 0.5 \cdot d_b)]$$

$$= \pi \cdot [36.00 - 2 \cdot (3.50 + 0.5 \cdot 1.128)] = 87.56 \text{ in}$$

$$\text{Dowel bar spacing} = \frac{87.56}{12} = 7.30 \text{ in}$$

$$\text{Dowel bar clear spacing} = 7.30 - 1.128 = 6.17 \text{ in.}$$

The basic development length  $\ell_{db}$  for a #9 bar is:

$$\ell_{db} = \frac{2.4 \cdot d_b \cdot f_y}{\sqrt{f'_c}} = \frac{2.4 \cdot 1.128 \cdot 60}{\sqrt{4}} = 81.22 \text{ in}$$

The modification factors to the development length are:

$$\lambda_{rl} = 1.0 \text{ for vertical bars}$$

$$\lambda = 1.0 \text{ for normal weight concrete}$$

$$\lambda_{er} = 1.0 \text{ taken conservatively assuming } A_{s\text{provided}} = A_{s\text{required}}$$

#### [5.11.2.1.2]

For determination of  $\lambda_{cf}$ :

The dowel bars are epoxy coated

$$3 \cdot d_b = 3.38 \text{ in} < \text{dowel bar clear cover}$$

$$6 \cdot d_b = 6.77 \text{ in} > \text{dowel bar clear spacing}$$

$$\text{Then } \lambda_{cf} = 1.5$$

#### [5.11.2.1.3]

For determination of  $\lambda_{rc}$ :

$$c_b = 3.70 \text{ in. (governed by } 0.5 \cdot \text{bar spacing)}$$

$$s = 3 \text{ in (spiral pitch)}$$

$n = 1$  (spiral crosses splitting plane)  
 $A_{tr} = 0.20$  (area of #4 spiral bar)

$$\text{Then } k_{tr} = \frac{40 \cdot A_{tr}}{s \cdot n} = \frac{40 \cdot 0.20}{3 \cdot 1} = 2.67$$

$$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}} = \frac{1.128}{3.70 + 2.67} = 0.18 < 0.4$$

So  $\lambda_{rc} = 0.4$

Then the development length  $\ell_d$  is:

$$\ell_d = \frac{\ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = \frac{81.22 \cdot (1.0 \cdot 1.5 \cdot 0.4 \cdot 1.0)}{1.0} = 48.73 \text{ in.}$$

The lap length for a Class B tension splice is governed by the smaller bar size, in this case the #8 column bar. The projection of the #9 dowel will be governed by the greater of the development length of the #9 dowel and the Class B lap for the #8 column bar.

For the #8 column bars:

$$\text{Clear cover } d_{colclr} = 2.00 + 0.50 = 2.50 \text{ in}$$

$$\begin{aligned} \text{Column bar circle diameter} &= \pi \cdot [d_{col} - 2 \cdot (d_{colclr} + 0.5 \cdot d_b)] \\ &= \pi \cdot [36.00 - 2 \cdot (2.50 + 0.5 \cdot 1.00)] \\ &= 94.25 \text{ in} \end{aligned}$$

$$\text{Column bar spacing} = \frac{94.25}{12} = 7.85 \text{ in}$$

$$\text{Column bar clear spacing} = 7.85 - 1.00 = 6.85 \text{ in}$$

The basic development length  $\ell_{db}$  for a #8 bar is:

$$\ell_{db} = \frac{2.4 \cdot d_b \cdot f_y}{\sqrt{f'_c}} = \frac{2.4 \cdot 1.00 \cdot 60}{\sqrt{4}} = 72.00 \text{ in}$$

The modification factors to the development length are:

$$\lambda_{rl} = 1.0 \text{ for vertical bars}$$

$$\lambda = 1.0 \text{ for normal weight concrete}$$

$$\lambda_{er} = 1.0 \text{ taken conservatively assuming } A_{sprovided} = A_{srequired}$$

For determination of  $\lambda_{cf}$ :

The column bars are epoxy coated

$$3d_b = 3.00 \text{ in} > \text{column bar clear cover}$$

$$6d_b = 6.00 \text{ in} < \text{column bar clear spacing}$$

$$\text{Then } \lambda_{cf} = 1.5$$

For determination of  $\lambda_{rc}$ :

$$c_b = 3.00 \text{ in. (governed by clear cover} + 0.5 \cdot d_b)$$

$$s = 3 \text{ in (spiral pitch)}$$

$$n = 1 \text{ (spiral crosses splitting plane)}$$

$$A_{tr} = 0.20 \text{ (area of \#4 spiral bar)}$$

$$\text{Then } k_{tr} = \frac{40 \cdot A_{tr}}{s \cdot n} = \frac{40 \cdot 0.20}{3 \cdot 1} = 2.67$$

$$\lambda_{rc} = \frac{d_b}{c_b + k_{tr}} = \frac{1.00}{3.00 + 2.67} = 0.18 < 0.4$$

$$\text{So } \lambda_{rc} = 0.4$$

Then the development length  $\ell_d$  is:

$$\ell_d = \frac{\ell_{db} \cdot (\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = \frac{72.00 \cdot (1.0 \cdot 1.5 \cdot 0.4 \cdot 1.0)}{1.0} = 43.20 \text{ in.}$$

The lap length for a Class B tension splice is:

**[5.11.5.3.1]**

$$1.3 \cdot \ell_d = 1.3 \cdot 43.20 = 56.16 \text{ in}$$

The Class B lap length for a #8 bar governs over the development length of a #9 bar.

Specify a 4'-9" lap length.

### **Dowel Bar Hook Development**

Verify that adequate embedment is provided for the dowel bars in the footing.

The basic development length  $\ell_{hb}$  for a #9 epoxy coated bar with a standard hook is:

**[5.11.2.4.1]**

$$\ell_{hb} = \frac{38.0 \cdot d_b \cdot f_y}{60.0 \cdot \lambda \cdot \sqrt{f'_c}} = \frac{38.0 \cdot 1.128 \cdot 60.0}{60.0 \cdot 1.0 \cdot \sqrt{4}} = 21.43 \text{ in}$$

Applicable development length modification factors are:

- $\lambda_{rc} = 0.8$  for side cover  $\geq 2.5$  inches and  $90^\circ$  hook extension cover  $\geq 2.0$  inches.
- $\lambda_{cf} = 1.2$  for epoxy coated bars.

The development length  $\ell_{dh}$  of the dowel with standard hook is:

$$\ell_{dh} = 21.43 \cdot 0.8 \cdot 1.2 = 20.57 \text{ in}$$

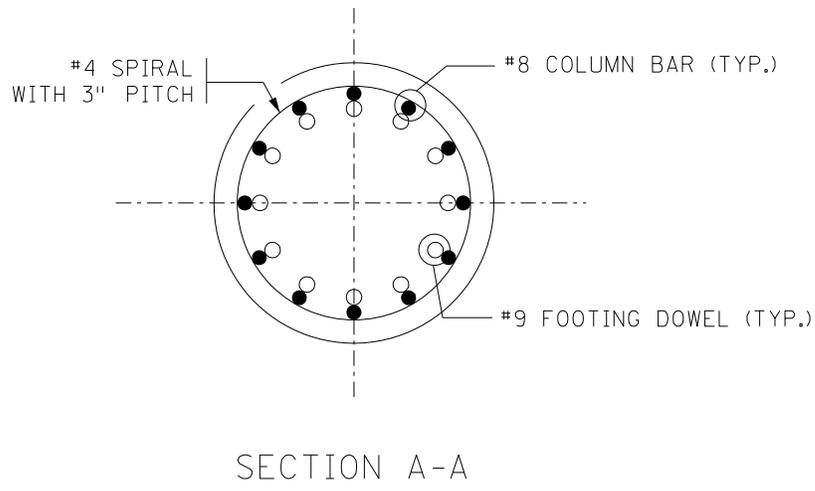
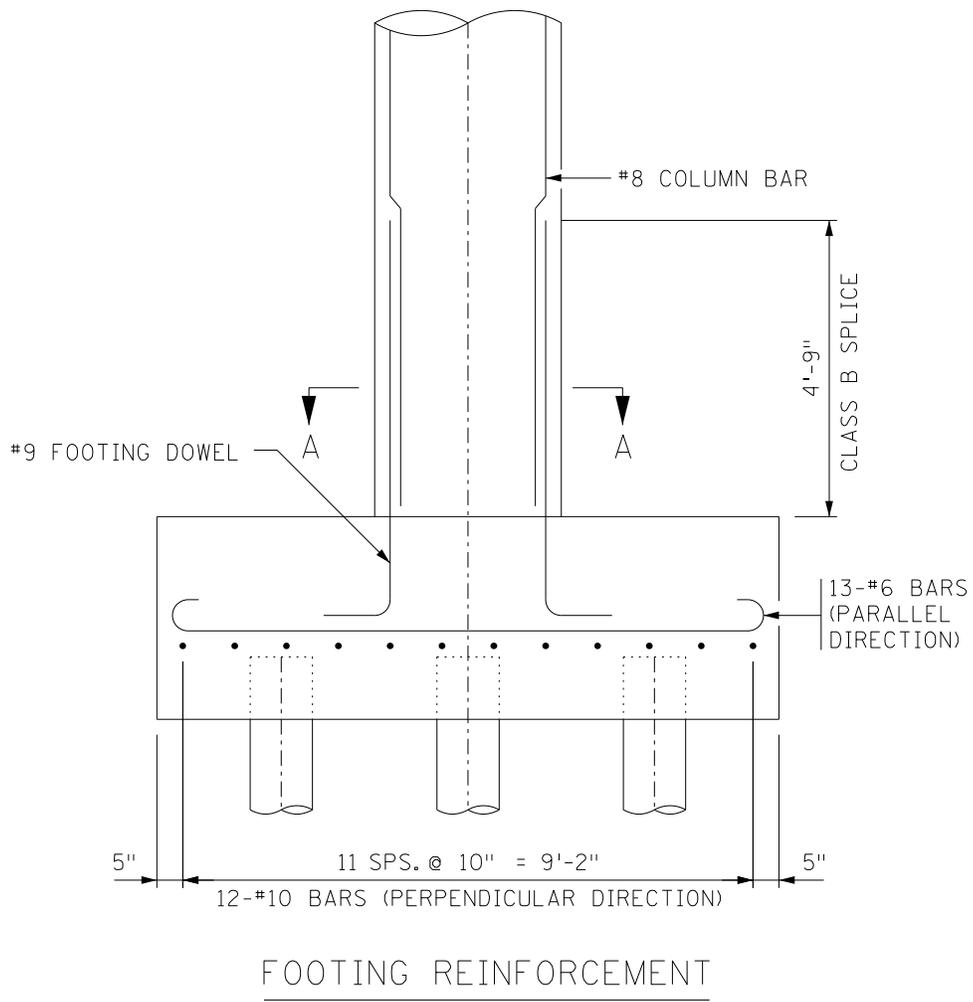
The embedment provided is:

$$\ell_{prov} = 54 - 12 - 1.27 - 0.75 = 39.98 \text{ in} > 20.57 \text{ in}$$

OK

**5. Summary**

The footing reinforcement is illustrated in Figure 11.4.3.18.



**Figure 11.4.3.18**

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**12. BURIED  
STRUCTURES**

Buried structures serve a variety of purposes. They are typically used for conveying water. At other times they are used to provide a grade separated crossing for pedestrian and bicycle traffic. A variety of structure and material types are used. The most prevalent types are pipes and box culverts. Buried structures with horizontal dimensions less than 10'-0" are not classified as bridges. Typically these smaller buried structures do not require extensive design and are selected from standard design tables. Buried structures with horizontal dimensions greater than or equal to 10'-0" are considered bridges and require a plan prepared by the Bridge Office. All box culverts require a Bridge Office prepared plan as well. In addition to pipes and box culverts, precast concrete arches, precast three-sided structures, and long-span corrugated steel structures are used as buried structures.

**[12.6.6]**

Buried structures carry vertical loads through a combination of internal capacity and soil arching around the structure; this is termed soil-structure interaction. The means by which a buried structure carries vertical load varies significantly between different structure types due to their relative stiffness. Concrete box culverts and rigid pipes are classified as rigid culverts and are assumed to carry the design loads internally with limited requirements or benefit of the soil. Flexible pipe structures (corrugated steel, thermoplastic, etc.) carry loads through soil-structure interaction. For this reason, material and installation requirements of the pipe and soil are well defined including trench or embankment conditions and backfilling and compaction procedures to ensure that the assumed soil-structure capacity is provided and that settlements are not excessive. AASHTO has developed empirical equations for different pipe types to allow for a simplified procedure that closely matches 3D soil-structure interaction models.

For special designs a 3D soil-structure model may be utilized in designing and detailing. This will require additional approvals and procedures to ensure the quality of the analysis and construction sequence. Approval of the State Bridge Design Engineer is required for use.

**12.1 Geotechnical  
Properties**

Typically, one or more soil borings will be obtained during the preliminary design process. Foundation recommendations based on field data and the hydraulic requirements will also be assembled during the preliminary design process. MnDOT Spec 2451 describes the excavation, foundation preparation, and backfill requirements for bridges and miscellaneous structures.

Maximum and minimum load factors for different load components should be combined to produce the largest load effects. The presence or absence of water in the culvert should also be considered when assembling load combinations.

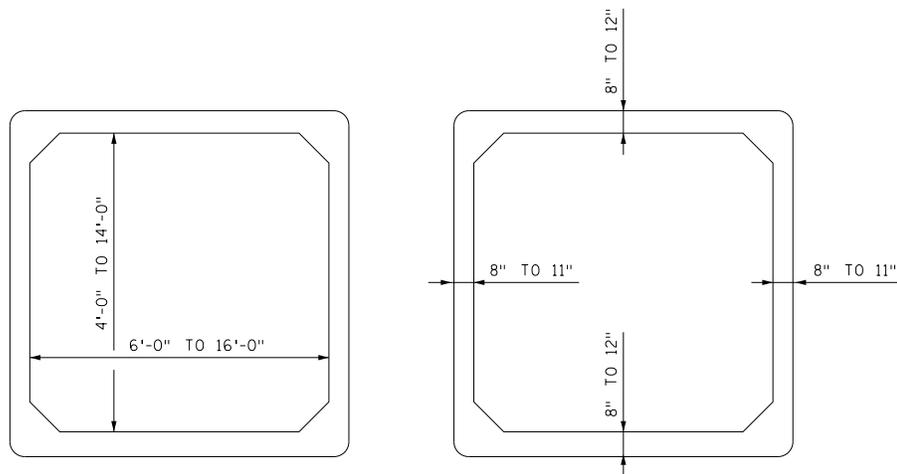
**12.2 Box Culverts**

Where pipe solutions are inappropriate, box culverts are the default buried structure type. Their larger openings are often required to provide adequate hydraulic capacity. Box culverts are also frequently used for pedestrian or cattle underpasses.

The reinforcement used in concrete box culverts can be either conventional bar reinforcement or welded wire fabric. Welded wire fabric has a yield strength slightly larger than conventional bar reinforcement (65 ksi versus 60 ksi).

**12.2.1 Precast Concrete Box Culverts**

Standard designs for precast concrete box culverts are available with spans varying from 6 to 16 feet and rises varying from 4 to 14 feet. Standard precast concrete box culverts are typically fabricated in 6 foot sections; however larger boxes are fabricated in 4 foot sections to reduce section weight. The designs utilize concrete strengths between 5 and 6 ksi and are suitable for fill heights ranging from less than 2 feet to a maximum of 25 feet. Box culverts outside of the standard size ranges must be custom designed. Figure 12.2.1.1 shows typical precast concrete box culvert dimensions.



**Figure 12.2.1.1**  
**Typical Precast Concrete Box Culvert Dimensions**

Each culvert size has three or four classes. Each class has specified wall and slab thicknesses, reinforcement areas, concrete strength, and fill

height range to which it applies. Shop drawing submittals for MnDOT approval will not be required when standard culvert sections are used.

The standard design tables are based on welded wire fabric reinforcement with a yield strength of 65 ksi and a concrete clear cover of 2 inches. MnDOT requires that actual clear cover be between 1.5 inches and 2 inches. Design information for welded wire reinforcement can be found at the Wire Reinforcement Institute website:

<http://www.wirereinforcementinstitute.org>

If conventional rebar is used, the steel area shown on the standard plan sheets needs to be increased 8% to account for the difference in steel yield strength (65 ksi/60 ksi). Also, crack control must be rechecked for the specific bar size and spacing used.

To prevent corrosion at the ends of welded wire fabric, nylon boots are required on the ends of every fourth longitudinal wire at the bottom of the form. Plastic spacers may be utilized in lieu of nylon boots when spaced at a maximum of 48 inches. The maximum allowable size of reinforcement bars is #6 and the maximum allowable size of welded wire is W23. A maximum of two layers of welded wire fabric can be used for primary reinforcement. If two layers are used, the layers may not be nested.

### **12.2.2 Cast-In-Place Concrete Box Culverts**

The first box culverts constructed in Minnesota were made of cast-in-place concrete. The performance of these structures over the years has been very good. Currently, most box culvert installations are precast due to the reduced time required for plan production and construction. Cast-in-place culverts continue to be an allowable option.

### **12.2.3 Design Guidance for Box Culverts**

#### **Material Properties**

Concrete Compressive Strength  $f'_c = 5$  ksi or 6 ksi

Steel Yield Strength  $f_y = 65$  ksi (welded wire fabric)

Steel Yield Strength  $f_y = 60$  ksi (rebar)

Reinforced Concrete Unit Weight  $\gamma_c = 0.150$  kcf

Soil Fill Unit Weight  $\gamma_s = 0.120$  kcf

Culvert Backfill Angle of Internal Friction  $\phi'_f = 30$  degrees

Water Unit Weight  $\gamma_w = 0.0624$  kcf

#### **Geometry**

The minimum wall thickness for all box culverts is 8 inches. The minimum slab thickness for culverts with spans of 6 to 8 feet is 8 inches. The

minimum top slab thickness is 9 inches, and the minimum bottom slab is 10 inches for all culverts with spans larger than 8 feet. The slab and/or wall thickness is increased when shear requirements dictate or the maximum steel percentages are exceeded. All standard box culverts have haunches that measure 12 inches vertically and horizontally.

### **Structural Analysis**

Various methods can be used to model culverts. Based on past experience, MnDOT prefers a 2-Dimensional (2D) plane frame model be used to analyze culverts. The model is assumed to be externally supported by a pinned support on one bottom corner and roller support on the other bottom corner. The stiffness of the haunch is included in the model. The model is assumed to be in equilibrium so external reactions to loads applied to the structure are assumed to act equal and opposite. This section will assume a 2D plane frame model when referring to modeling, applied loads, and self-weight.

### **Self Weight (DC)**

The self-weight of the top slab must be resisted by the top slab. The benefit of axial compression from the self-weight of the top slab and walls is not included in the analysis. The top slab, wall, and all haunch weights are applied to the bottom slab as an upward reaction from the soil in an equivalent uniform pressure. The bottom slab weight is not applied in the model because its load is assumed to be directly resisted by the soil.

### **Earth Vertical (EV)**

The design fill height is measured from the top surface of the top slab to the top of the roadway or fill. The design fill height is denoted by the abbreviations of  $H$  or  $D_E$  depending on the equation used. Earth vertical loads refer to soil and pavement loads above the culvert and in adjacent regions slightly outside the span of the culvert based on the soil-structure interaction factor. Culvert walls are assumed to be frictionless, so no vertical component of the earth horizontal resultant force is considered.

#### **[12.11.2.2.1]**

The soil-structure interaction factor ( $F_e$ ) is used to adjust the vertical earth load carried by the culvert. It is intended to approximate the arching effects of some of the overburden soil to adjacent regions slightly outside the span of the culvert and account for installation conditions. Culverts placed in trench conditions need to carry less vertical load than those constructed in embankment conditions, because the consolidated material in the adjacent trench walls is typically stiffer than new embankment material. Conservatively assume culverts are installed in embankment conditions.

The factor is:

$$[12.11.2.2.1-2] \quad F_e = 1 + 0.20 \cdot \frac{H}{B_c}$$

where:

H = Depth of backfill (ft)

B<sub>c</sub> = Outside width of culvert (2 · sidewall thickness + span) (ft)

### **Earth Horizontal (EH)**

**[3.11.5.5]**

For design and analysis purposes, the equivalent fluid method is used. The maximum for lateral earth pressure on the walls based on at rest pressure is 0.060 kcf.

**[3.11.5.2-1]**

This is computed by taking  $k_o \cdot \gamma_s$ , where:

$$k_o = 1 - \sin(\phi'_f) = 1 - \sin(30^\circ) = 0.5$$

The resultant earth horizontal force is assumed to act perpendicular to the culvert walls. For maximum force effects, use a strength limit state load factor of 1.35 and a service limit state load factor of 1.0.

**[3.11.7]**

For minimum force effects, the condition of submerged soil pressure acting on the walls is taken as one-half of the earth weight acting on the outside walls, or 0.030 kcf. Use a strength limit state load factor of 0.9 and a service limit state load factor of 1.0.

**[3.7.1]**

### **Water (WA)**

Designers need to consider two loading conditions: 1) The culvert is full of water, and 2) the culvert is empty.

**[3.6.1.2.2]**

**[3.6.1.2.3]**

**[3.6.1.3.3]**

### **Design Vehicular Live Load (LL)**

The approximate strip method is used for design with the 1 foot wide design strip oriented parallel to the span. The design live loads applied to the top slabs of box culverts include the HL-93 truck and tandem loads for box culverts of any span length. For box culverts with spans of 15 feet or greater lane loads are also applied to the top slabs of box culverts. This practice is consistent with previous versions of the AASHTO Standard Specifications for Highway Bridges.

**[3.6.1.2.4]**

**[12.11.2.1]**

### **Design Lane Loads**

The design lane load consists of a load of 0.64 klf uniformly distributed over an area of 1 foot (parallel to the culvert span) by 10 feet perpendicular to the culvert span.

**Tire Contact Area****[3.6.1.2.5]**

The tire contact area of a wheel consisting of one or two tires is assumed to be a single rectangle, whose width is 20 inches and whose length is 10 inches. The tire pressure is assumed to be uniformly distributed over the rectangular contact area on continuous surfaces.

**[3.6.1.1.2]****[C12.11.2.1]****One or Two Lane Loading and Multiple Presence Factor (MPF)**

Design box culverts for a single loaded lane with a multiple presence factor of 1.2. MnDOT investigated several live load cases with several box culvert spans at different fill heights and found the live load intensity of 2 lanes with a MPF of 1.0 controlled over a single lane with a multiple presence factor of 1.2 at fill heights of 6.5 feet and greater. However, the maximum live load intensity increase as a percentage of the total load is very small. Based on these findings and the commentary in AASHTO Article C12.11.2.1, multiple loaded lanes are not considered in box culvert design.

**[3.6.2.2]****Dynamic Load Allowance (IM)**

The dynamic load allowance (IM) for culverts and other buried structures is reduced based on the depth of fill over the culvert. AASHTO LRFD requires that IM be considered for fill heights of up to 8 ft. The equation to calculate the dynamic load allowance is as follows:

$$IM = 33 \cdot (1.0 - 0.125 \cdot D_E) \geq 0\% \text{ (for strength and service limit states)}$$

where:

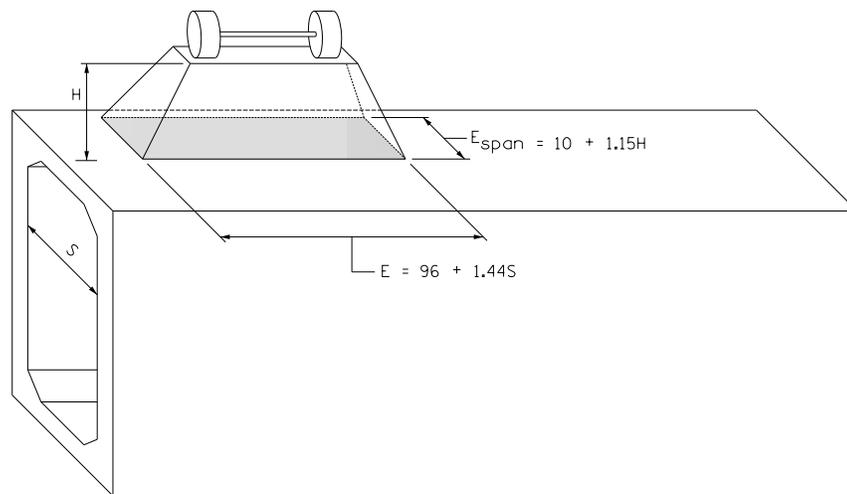
$D_E$  = the minimum depth of earth cover above the structure (ft)

**[3.6.1.3]****Live Load Influence Depth**

Include live load for all fill heights.

**[12.11.2.1]****[4.6.2.10]****Live Load Distribution With Less Than 2 Feet of Fill**

Most box culverts are designed assuming traffic travels parallel to the span. In that scenario, when the depth of fill measured from the top of the roadway or fill to the top of the top slab is less than 2 feet, distribute the design truck or design tandem loads according to AASHTO 4.6.2.10.2 (Case 1: Traffic Travels Parallel to Span). If traffic travels perpendicular to the span, design according to AASHTO 4.6.2.1. Traffic traveling perpendicular to the span is not covered in this manual.



**Figure 12.2.3.1**  
**Traffic Traveling Parallel to Span (Less than 2 feet of fill)**

The truck axle loads are considered to be uniformly distributed over a rectangular area equal to  $E \cdot E_{span}$ , as shown in Figure 12.2.3.1, where:

**[4.6.2.10.2-1]**

$$E = 96 + 1.44 \cdot S$$

**[4.6.2.10.2-2]**

$$E_{span} = L_T + LLDF \cdot (H)$$

where:

E = equivalent distribution width perpendicular to span (in)

S = clear span (ft)

$E_{span}$  = equivalent distribution length parallel to span (in)

$L_T$  = length of tire contact area parallel to span (in)

LLDF = 1.15, factor for distribution of live load through depth of fill

H = depth of fill from top of culvert to top of pavement (in)

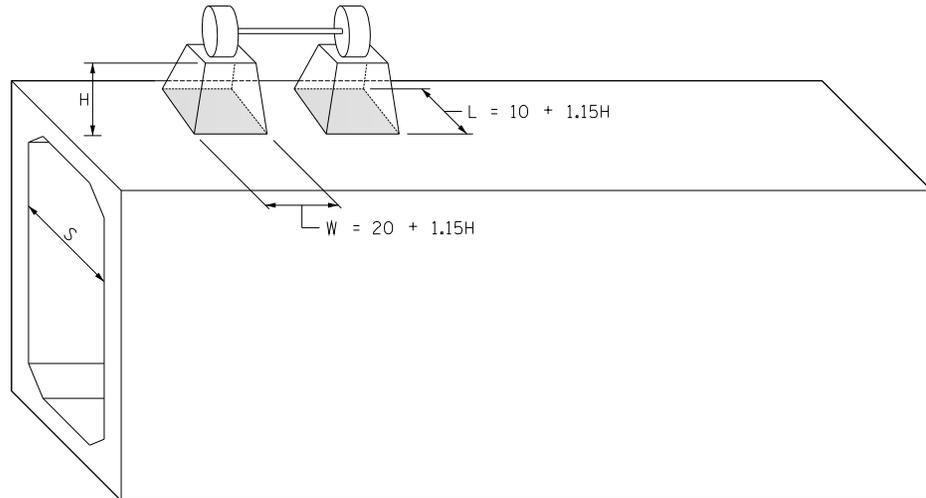
**[4.6.2.10.4]**

Box culverts with fill heights less than 2 feet require a distribution slab. No structural benefit from the distribution slab is considered during design, other than satisfying AASHTO requirements for shear transfer across joints.

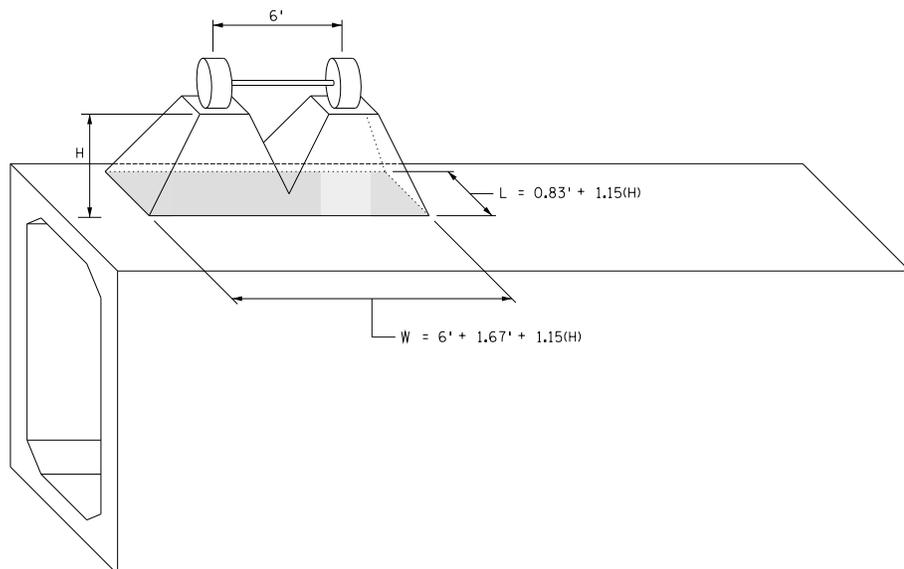
**Live Load Distribution With 2 Feet of Fill or Greater**

Where the depth of fill exceeds 2 feet, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area and increased by either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases. Note that the tables in the MnDOT standard plans use 1.15. MnDOT has not adopted the LLDF's as revised in the AASHTO 2013 Interim Revisions, Article 3.6.1.2.6.

The load distribution is shown in Figure 12.2.3.2 for cases where the distributed load from each wheel is separate. Figure 12.2.3.3 shows the areas overlapping. In those cases, the total load will be uniformly distributed over the entire area. In Figure 12.2.3.2, H is measured in inches. In Figure 12.2.3.3, H is measured in feet.



**Figure 12.2.3.2**  
**Traffic Traveling Parallel to Span (2 feet of fill or greater)**

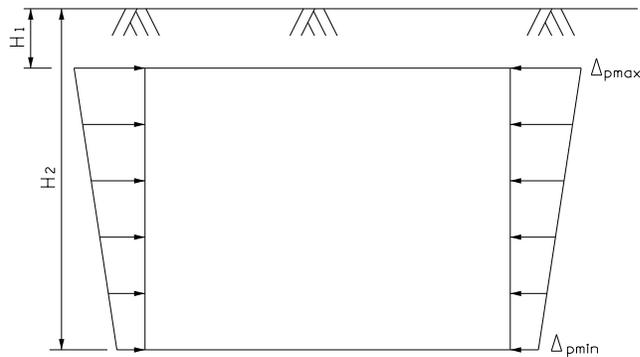


**Figure 12.2.3.3**  
**Traffic Traveling Parallel to Span**  
**(2 feet of fill or greater showing load projection overlap)**

**Live Load Surcharge (Approaching Vehicle Load)**

**[3.11.6.4]**

AASHTO requires that a live load surcharge be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. MnDOT uses a modified form of AASHTO Article 3.11.6.4 to compute the approaching vehicle load. A trapezoidal pressure distribution is assumed with the maximum pressure  $\Delta_{pmax}$  at the top of the box culvert and the minimum pressure  $\Delta_{pmin}$  at the bottom of the box culvert. The live load surcharge is only to be applied to one wall of the culvert. For simplification of the analysis, MnDOT applies an equal and opposite reaction to the other wall.



**Figure 12.2.3.4  
Live Load Surcharge**

This methodology more closely approximates a Boussinesq load distribution than assuming a rectangular distribution with an at rest coefficient of lateral earth pressure. Use AASHTO, Equation 3.11.6.4-1 to compute the horizontal earth pressures ( $\Delta_{pmax}$  and  $\Delta_{pmin}$ ) assuming an active coefficient of lateral earth pressure ( $k_a = 0.33$ ).

$$\Delta_p = k_a \cdot \gamma_s \cdot h_{eq}$$

where:

- $\Delta_p$  = horizontal earth pressure due to live load surcharge (ksf)
- $k_a$  = coefficient of lateral earth pressure
- $\gamma_s$  = total unit weight of soil
- $h_{eq}$  = equivalent height of soil for vehicular load (ft), from AASHTO Table 3.11.6.4-1

For calculating  $\Delta_{pmin}$ , determine  $h_{eq}$  based on the distance from the top of the top slab to the top of the pavement or fill ( $H_1$ ). For calculating  $\Delta_{pmax}$  determine  $h_{eq}$  based on the distance from the bottom of the bottom slab to the top of the pavement or fill ( $H_2$ ). Use linear interpolation for intermediate heights.

[3.4.1]

**Limit States and Load Combinations**

[12.5]

Design for the Strength I and Service I limit states. Evaluation of extreme event and fatigue limit states is unnecessary because culvert design is not governed by these limit states.

**Load Combinations**

The following load combinations were developed by varying the Strength I and Service I load factors in order to maximize moments and shears for the various box culvert members. At a minimum, consider the following load cases:

**Strength Limit States:**

Ia. Maximum vertical load and maximum horizontal load:

$$1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + (1.35)(1.05)EH_{max} + 1.75LS$$

Ib. Maximum vertical load and minimum horizontal load:

$$1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + 1.00WA + (0.9/1.05)EH_{min}$$

Ic. Minimum vertical load and maximum horizontal load:

$$0.90DC + (0.90/1.05)EV + (1.35)(1.05)EH_{max} + 1.75LS$$

**Service Limit States:**

Ia. Maximum vertical load and maximum horizontal load:

$$1.00DC + 1.00EV + 1.00(LL+IM) + 1.00EH_{max} + 1.00LS$$

Ib. Maximum vertical load and minimum horizontal load:

$$1.00DC + 1.00EV + 1.0(LL+IM) + 1.00WA + 1.00EH_{min}$$

Ic. Minimum vertical load and maximum horizontal load:

$$1.00DC + 1.00EV + 1.00EH_{max} + 1.00LS$$

[1.3.4]

[12.5.4]

Use a value of 1.0 for all load modifiers ( $\eta$ ) for box culvert design, except for earth EV and EH loads, EV & EH where  $\eta_R = 1.05$  is used due to the lack of redundancy.

[C12.11.3]

**Axial Thrust**

Do not consider the benefit of axial thrust in the design of box culverts for the strength limit state. It may be used in the service limit state crack control check.

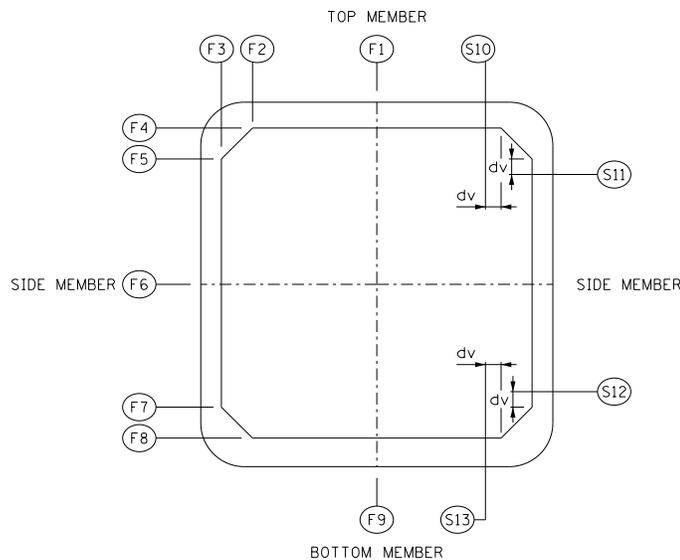
[12.11.4.2]

**Flexure**

Flexural reinforcement is designed for positive and negative moment at all design locations (see Figure 12.2.3.5). The flexural resistance factor,  $\phi_f$ , is 1.0 for precast concrete. Reinforcing areas, shown in Figure 12.2.3.6, are selected based on the following:

[12.5.5]

- As1 is based on the negative moment requirements in the side wall and in the outside face of the top slab and bottom slab
- As2 is based on the positive moment in the top slab.
- As3 is based on the positive moment in the bottom slab.
- As4 is based on the positive moment in the side
- As7 is based on the negative moment at Section F1
- As8 is based on the negative moment at Section F9.

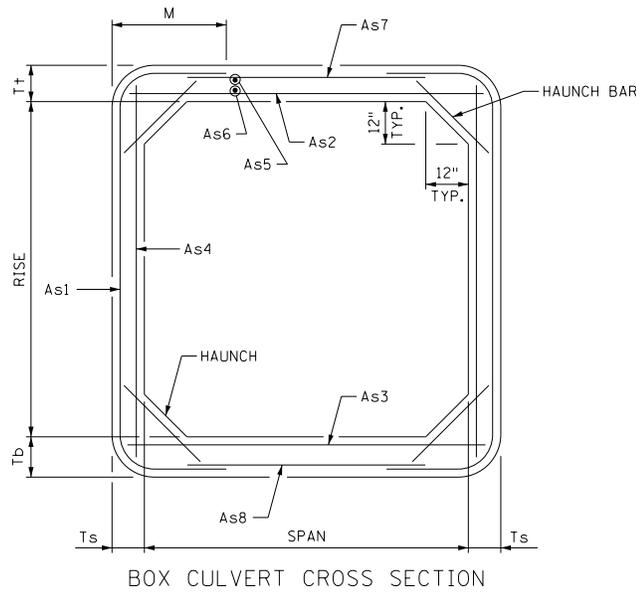


F (1-9) : FLEXURE DESIGN LOCATIONS

S (10-13) : SHEAR DESIGN LOCATIONS

dv : DISTANCE FROM THE TIP OF THE HAUNCH TO THE CRITICAL SHEAR SECTION

**Figure 12.2.3.5**  
**Box Culvert Flexure and Shear Design Locations**



**Figure 12.2.3.6**  
**Box Culvert Reinforcement**

**[5.7.3.4]**  
**[C12.11.3]**

**Crack Control**

Restrict the stress in the reinforcement to 60% of the yield strength. For welded wire fabric, assume a maximum spacing of 4 inches. Check crack control using the Class II exposure condition ( $\gamma_e=0.75$ ). Compute the tensile stress in the steel reinforcement at the service limit state using the benefits of axial thrust as shown in AASHTO equation C12.11.3-1. Fabricators have discretion in choosing wire spacing, but the spacing cannot exceed 4 inches.

**[5.5.4.2]**  
**[5.7.2.1]**  
**[12.5.5]**

**Maximum Reinforcement**

The standards and typical designs use a resistance factor of 1.0 with a section that is tension-controlled. Special designs may require a reduced resistance factor.

Reinforcement is limited to  $0.6\rho_b$ . This ensures that the reinforcement is not too congested, allowing for easier and more efficient fabrication.

**[5.10.8]**  
**[9.7.3.2]**  
**[12.11.4.3.2]**

**Minimum Reinforcement**

MnDOT requires reinforcement in all slabs and walls in both directions on both faces regardless of fill height. In top and bottom slabs for all fill heights, use  $0.002 \times b \times h$  as the minimum primary reinforcement denoted as As7 and As8. Distribution reinforcement is not needed, since a distribution slab is required for all boxes with less than 2.0 feet of fill.

A minimum amount of reinforcement is required to be placed in each face in each direction in the top and bottom slabs and walls for all box sections regardless of cover. The MnDOT minimum value for this reinforcement is  $0.06 \text{ in}^2/\text{ft}$ , which is greater than the AASHTO minimum.

**[5.13.3.6]**

**Shear Critical Section**

**[C5.13.3.6.1]**

Because of the 1:1 slope of the haunch, the critical section for shear may be taken at  $d_v$  past the tip of the haunch.

**[5.8]**

**Shear in Slabs of Box Culverts with Less Than 2 Feet of Fill and Walls of Box Culverts at All Fill Heights**

**[12.5.5]**

For top slabs of boxes with less than 2 feet of fill and walls of boxes at all fill heights calculate the shear resistance using the greater of that computed using the "Simplified Procedure for Nonprestressed Sections" given in AASHTO LRFD Article 5.8.3.4.1 and the "General Procedure" given in AASHTO Article 5.8.3.4.2.

**[5.14.5.3]**

**Shear in Slabs of Box Culverts with 2 Feet of Fill or Greater**

**[12.5.5]**

For top and bottom slabs of boxes with 2 feet of fill or greater calculate the shear resistance using the shear provisions specific to slabs of box culverts.

For slabs of boxes with thicknesses greater than 12 inches, contact the MnDOT Bridge Standards Unit for shear provisions.

**[C12.5.3]**

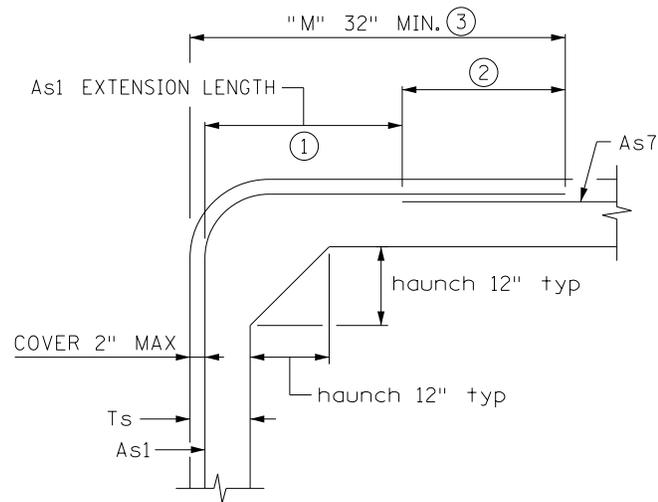
**Fatigue**

Fatigue is not considered in the design of buried structures.

**[5.11.2.5.2]**

**Development Lengths**

To ensure reinforcement continuity, proper development length is required. See Figure 12.2.3.7 for extension of  $A_{s1}$  into the top or bottom slab. For constructability, make the bent legs on  $A_{s1}$  the same length on the top and bottom. This length is typically calculated based on the bottom slab. MnDOT uses AASHTO LRFD equation 5.11.2.5.2-1 to calculate development lengths in box culverts. In some cases,  $A_{s1}$  is needed to resist shear. In these cases,  $A_{s1}$  should be developed past  $d_v$  from the tip of the haunch.



- ① As1 EXTENSION LENGTH - DISTANCE TO THE POINT WHERE THE NEGATIVE MOMENT CAN BE RESISTED BY As7 OR As8.
- ② GREATER OF DEVELOPMENT LENGTH OF As1 OR THE LAP SPLICE LENGTH OF As7.
- ③ "M" = 2" (COVER) + ① + ②  
 MINIMUM "M" DISTANCE =  $T_s + \text{HAUNCH (12")} + \text{MAX}(d_o, d_v) + 6"$

**Figure 12.2.3.7**  
**Box Culvert Reinforcement Development Length**

Unless the specific size of welded wire fabric to be used by the fabricator is known, use the largest size that can provide the area required in one mat. If two mats are required, use a W23 for the development length calculation.

**Aprons**

Precast apron segments are provided for each size of barrel. There are four different details relating the culvert's skew to the roadway above.

Culvert Skew Range	Apron Skew
0° to 7½°	0°
7½° to 22½°	15°
22½° to 37½°	30°
37½° to 45°	45°*

\* Boxes with spans of 16 feet or greater have a maximum apron skew angle of 30°. All other boxes have a maximum apron skew of 45°.

Based on past practice, lateral soil pressure of 0.060 ksf is used for the apron design except for the 45° skew aprons which are designed with a 0.075 ksf pressure on the longer length wall. MnDOT also requires additional extra strong ties between the barrel and first end section, and between the first and second end sections on the high fill side only for 45° skew aprons over 6 feet high. Conventional ties can be used on aprons between multiple boxes and on the low fill side of the apron. Additional ties are required to resist unequal pressures on opposite sides of the skewed apron. See the culvert standards Figure 5-395.110(A) for more information.

### **Software**

Various commercially available off-the-shelf software programs have been developed to analyze and design precast box culverts. These software programs can be used to assist in the design of precast box culverts provided that the parameters, modeling methods, AASHTO LRFD code provisions and MnDOT code modifications specified in this manual are compatible with the software. In some instances, it may be easier to develop custom software or spreadsheets depending on the differences between the available software and the AASHTO and MnDOT practices detailed in this manual. Any piece of software is subject to the Design QC/QA Process outlined in Section 4.1.

### **District Box Culvert Request**

Figure 12.2.3.8 shows a typical box culvert request memo from a District.

### **Design Example**

Refer to Section 12.5 for a 10 ft x 10 ft precast concrete box culvert design example.



Minnesota Department of Transportation

MEMO

Transportation District #7

2151 Bassett Drive  
Mankato MN 56001-6888

Office Tel: 507-304-6100  
Fax No: 507-304-6119

Date: 06-27-2012

To: Kevin L. Western, Bridge Design Engineer  
Mn/DOT Bridge Office • MS 610  
3485 Hadley Avenue North • Oakdale, MN 55128-3307

From: Scott Morgan, District Hydraulics Engineer

Subject: S.P. 7201-112

Please prepare a design for concrete box culvert 8045. Tabulated below and attached is the information required to prepare plans. The letting date for this project is 03/22/2013. Please submit completed plans to this office before 09/15/2012. If you determine you are unable to meet the above deadline, please contact me.

State Project No. <b>7201-112</b>		Func. 2 Work Authority: <b>T 71559</b> (charge ID)	
Location Description <sup>(1)</sup> : <b>0.3 MI S OF GREEN ISLE</b>			
Reference: <b>14.091</b>		Station: <b>741+71.47</b>	
Section: <b>13</b>	Range: <b>R 27 W</b>	Township: <b>T 114 N</b>	Twp. Name: <b>GREEN ISLE</b>
Stream Crossing: <b>Sibley County Ditch No. 29</b>		County: <b>SIBLEY</b>	
Structure Type: <b>Box Culvert</b>			
New Structure Number: <b>72X04</b>		Existing Structure Number: <b>8045</b>	
Number of Barrels: <b>1</b>	Opening Width: <b>16 ft.</b>	Opening Height: <b>8 ft.</b>	Space btwn. Precast Barrels: <b>NA</b>
Depth of Cover <sup>(2)</sup> : <b>10.75 ft.</b>	Skew Angle: <b>22.5°</b>		End Sections: <b>Pre-Cast</b>
Inlet Elevation (new structure): <b>980.13</b>		Outlet Elevation (new structure): <b>979.63</b>	
Inlet Elevation (inplace structure): <b>982.20</b>		Outlet Elevation (inplace structure): <b>980.82</b>	
Extension Distances from End of Inplace: <b>NA</b>			
Plans Requested: <b>Precast Only</b>			

(1) T.H. 5 over Ditch No. 29, 0.26 Mi. SW of Jct. T.H. 5 & T.H. 25 (modify as necessary)  
 (2) Minimum fill over culvert rounded up to the next 1/4 ft.

Attachments: Sketch of Cross Section  
Sketch of Plan View

Cc: Angel Staples, Bridge Office Design Unit Leader  
Dustin Thomas, Bridge Office South Region Construction

**Figure 12.2.3.8**  
**District Box Culvert Request Memo Example**

**12.3 Arched &  
Three-Sided  
Structures**

Arched or three-sided precast concrete structures offer an alternative to single or multiple barrel box culvert structures. These structures can be constructed rapidly, thus minimizing road closure time, and they allow for a natural stream bottom. Potential applications include pedestrian underpasses and stream crossings where the waterway opening requirements are on the low end of a conventional bridge but are at the high end of box culvert capabilities. As with all structure type selections, the designer should consider speed of construction and economics, including cost comparisons to cast-in-place structures or multiple barrel precast concrete box culverts.

**12.3.1 Three-Sided  
Precast Concrete  
Structures**

There are two types of three-sided bridge structures: arch top and flat top. The design of such structures shall be in conformance with the AASHTO LRFD Bridge Design Specifications and the current Three-Sided Structures Technical Memorandum. The design methods vary between suppliers. The technical memorandum contains guidance for design, submittal requirements, material specifications, construction quality assurance, and the MnDOT Bridge Office review and approval process for use of three-sided structures.

In general, precast three-sided structures may be used where:

- A. Design span is less than or equal to 42 feet. Larger spans may be considered on a case-by-case basis, but only with prior approval of the Bridge Design Engineer. Span is measured from inside face of sidewalls along the longitudinal axis of the unit;
- B. Rise is less than or equal to 13 feet. Rise is measured from top of footing/pedestal wall to bottom of top slab;
- C. Fill height is less than or equal to 10 feet but is greater than or equal to 3 feet. Fill heights larger than 10 feet may be considered on a case-by-case basis, but only with prior approval of the Bridge Design Engineer;
- D. Skew is less than 30°;
- E. No foundation limitations exist such as unusually weak soil;
- F. No site access limitations exist for transporting and erecting the three-sided structures;

G. Clogging from debris or sediment precludes the use of multiple barrel structures.

Since these are vendor supplied structures, their final structural design occurs after the award of the construction contract. The time required for final design and the subsequent review/approval periods impact the total contract length.

This technical memorandum can be viewed at the following web site:

<http://techmemos.dot.state.mn.us/techmemo.aspx>

The list of pre-qualified suppliers for three-sided bridge structures is available at the Bridge Office website:

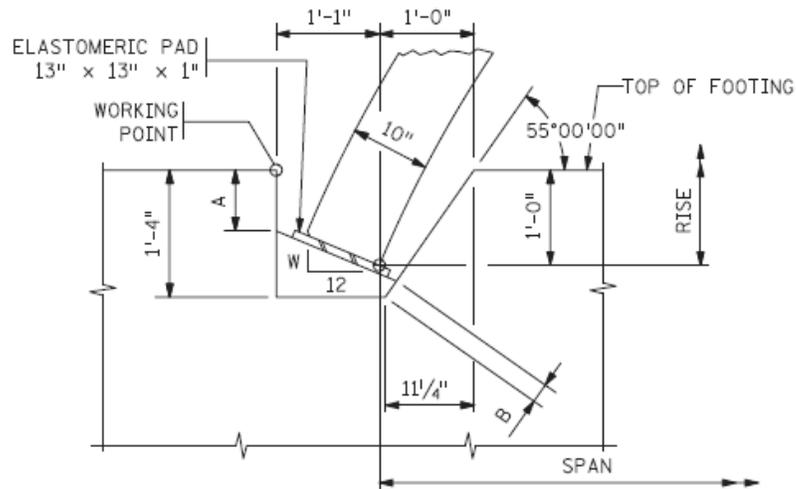
<http://www.dot.state.mn.us/products/bridge/3sidedprecastconc.html>

### ***12.3.2 Precast Concrete Arch Structures***

Sample plan sheets for the design of buried precast concrete arch structures are available from the MnDOT Bridge Standards Unit.

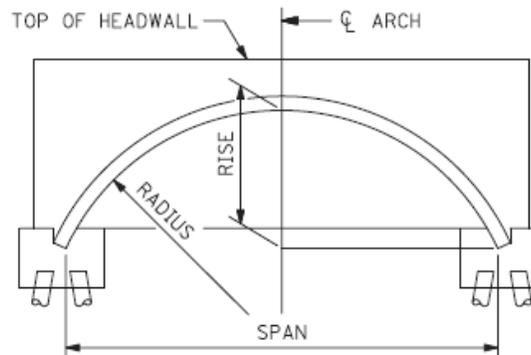
Figure 12.3.2.1 contains standard geometric information for spans between 24'-0" and 43'-11".

The minimum fill height is 1'-6" at the low edge of pavement at the crown of the arch.



**SECTION THROUGH FOOTING NOTCH**

ACTUAL SPAN	W	A	B	WORKING POINT TO WORKING POINT
24'-0"	5.00	7 <sup>5</sup> / <sub>8</sub> "	2 <sup>1</sup> / <sub>2</sub> "	26'-2"
25'-3 <sup>5</sup> / <sub>8</sub> "	2.85	10"	2 <sup>7</sup> / <sub>8</sub> "	27'-5 <sup>5</sup> / <sub>8</sub> "
32'-3"	5.95	6 <sup>5</sup> / <sub>8</sub> "	2 <sup>1</sup> / <sub>4</sub> "	34'-5"
34'-4 <sup>5</sup> / <sub>8</sub> "	3.72	9"	2 <sup>3</sup> / <sub>4</sub> "	36'-6 <sup>5</sup> / <sub>8</sub> "
40'-2 <sup>1</sup> / <sub>2</sub> "	8.36	4 <sup>1</sup> / <sub>8</sub> "	1 <sup>7</sup> / <sub>8</sub> "	42'-4 <sup>1</sup> / <sub>2</sub> "
43'-11 <sup>3</sup> / <sub>8</sub> "	5.92	6 <sup>3</sup> / <sub>4</sub> "	2 <sup>1</sup> / <sub>4</sub> "	46'-1 <sup>3</sup> / <sub>8</sub> "



**ELEVATION**  
(PILE FOOTING SHOWN)

NOMINAL SPAN	ACTUAL SPAN	RISE	RADIUS	AREA SQ. FT. ①	NET AREA SQ. FT. ②
24'-0"	24'-0"	8'-0"	13'-0"	139	117
25'-4"	25'-3 <sup>5</sup> / <sub>8</sub> "	10'-0"	13'-0"	188	165
32'-3"	32'-3"	10'-0"	18'-0"	232	202
34'-5"	34'-4 <sup>5</sup> / <sub>8</sub> "	12'-8"	18'-0"	320	288
40'-3"	40'-2 <sup>1</sup> / <sub>2</sub> "	10'-6"	24'-6"	296	258
43'-11"	43'-11 <sup>3</sup> / <sub>8</sub> "	13'-8"	24'-6"	430	388

① AREA UNDER ARCH ABOVE A LEVEL LINE AT LOWEST POINT OF ARCH.

② AREA UNDER ARCH ABOVE A LEVEL LINE AT TOP OF FOOTING.

**Figure 12.3.2.1**  
**Precast Concrete Arch Structure Geometric Data**

**12.3.3 Scour  
Protection  
Guidelines**

The following guidelines are provided for the design and installation of scour protection for arch or 3-sided bridge footings.

There are several options available for protection of the footings against scour. These options include rock riprap, concrete bottom, piling supported footings, and spread footings keyed into bedrock. The preferred option will depend on a number of factors including:

- Foundation design
- Stream bed material
- Scour potential
- Velocity of flow
- Environmental considerations such as fish migration
- Economics

The foundation design will depend on the type and allowable bearing capacity of the soil, the height of fill, and the proximity of bedrock. Scour should be considered during foundation design. Sub-cut unstable material below spread footings and replace it with granular backfill or a lean concrete. Due to the difficulty of achieving adequate compaction in wet conditions, the maximum depth of sub cutting for this purpose is 2 feet. A pile footing should be used if the depth of unstable material below a footing is greater than 2 feet.

Four standard designs for scour protection for concrete arch structures have been assembled. The appropriate design is selected based on the average velocity through the structure for the 100-year flood. A more recurrent flood event should be used if it results in a faster average velocity through the structure.

**Design 1 Scour Protection**

The average velocity for the 100 year flood must be no greater than three feet per second, and for the 500-year flood no greater than five feet per second. Use of 12 inch Class II riprap with 6 inch granular filter or geotextile filter is required.

- Option 1 (Figure 12.3.3.1, left side)  
The riprap may be placed on a slope of 1:2.5 maximum. Cover to the bottom of footing shall be 6 feet minimum measured perpendicular to the slope. The riprap shall be toed in vertically 2 feet minimum. The bottom of footing shall be at or below the channel bottom.

- Option 2 (Figure 12.3.3.1, right side)

The riprap may be placed horizontally on the channel bottom. Cover to the bottom of footing shall be 4'-6" minimum. The riprap shall extend a minimum of 10 feet from edge of structure and be toed in vertically a minimum of 2 feet.

#### **Design 2A Scour Protection**

The average velocity for the 100-year flood must be less than 5.5 feet per second, and for the 500-year flood less than 6.5 feet per second. Use of 24" Class IV riprap with 12" granular filter or geotextile filter is required.

- Option 1 (Figure 12.3.3.2, upper left side)

The riprap may be placed on a slope of 1:2.5 maximum. It shall extend across the entire width of the structure. Cover to the bottom of the footing shall be 6 feet minimum measured perpendicular to the slope. The bottom of footing shall be 2 feet minimum below the channel bottom.

- Option 2 (Figure 12.3.3.2, upper right side)

The riprap may be placed horizontally on the channel bottom. Cover to the bottom of footing shall be 6 feet minimum. The riprap shall extend a minimum of 10 feet from edge of footing and be toed in vertically a minimum of 2 feet.

#### **Design 2B Scour Protection** (Figure 12.3.3.2, lower right side)

The average velocity for the 100-year flood must be no greater than 5.5 feet per second. The average velocity for the 500-year flood must be no greater than 6.5 feet per second. The area for calculating the average velocity of the 100-year flood shall be Area, "A" which is bounded by the channel bottom and the water surface. The area for calculating the 500-year flood shall be Area "A" plus Area "B", where Area "B" is bounded by the channel bottom and the 500-year flood scoured channel bottom. The toe of riprap shall extend 2 feet min. beyond the bottom of Area "B". This toe shall have a minimum thickness of 2 feet. Cover to the bottom of footing shall be 6 feet minimum measured perpendicular to the slope. The bottom of footing shall also be at or below the bottom of Area "B".

#### **Design 3 Scour Protection**

- Option 1 (Figure 12.3.3.3, left side)

Articulated concrete with geotextile backing may be placed on a maximum slope of 1:2.5 with a minimum cover of 4'-6" to bottom of footing measured perpendicular to the articulated concrete. The average velocity for the 100-year flood must be no greater

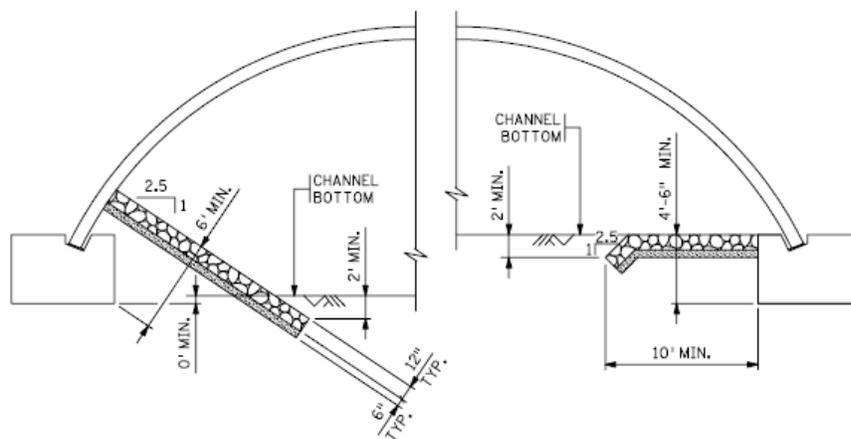
than 7.5 feet per second. For higher velocities, contact the Bridge Office.

- Option 2 (Figure 12.3.3.3, right side)  
A reinforced concrete floor placed horizontally only with 4'-6" minimum cover to bottom of footing may be used. The same velocity constraints as for Option 1 apply.

**Design 4 Scour Protection**

- Option 1 (Figure 12.3.3.4, left side)  
If footings are on piling, riprap shall be placed on a slope of 1:2.5 maximum. The bottom of footing shall be at or below the channel bottom.
- Option 2 (Figure 12.3.3.4, right side)  
If footings are on hard bedrock, they shall be keyed in a minimum of 1 foot.

These guidelines are anticipated to cover most cases, however, there may be factors such as high natural channel velocity, dense hardpan channel bottom, historical evidence of no scour on the in place structure or other pertinent data that can be considered when designing scour protection for the concrete arch structures. Exceptions to these guidelines must be approved by the Hydraulics Engineer.



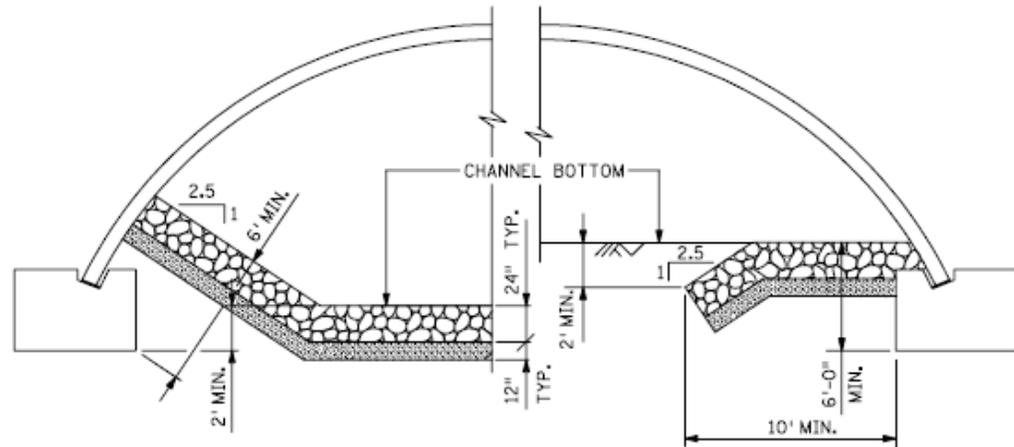
DESIGN 1 SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

12" CLASS II RIPRAP  
6" GRANULAR FILTER OR GEOTEXTILE FILTER

AVERAGE VELOCITY THROUGH STRUCTURE:  
 $V_{100} \leq 3$  f.p.s.  
 $V_{500} \leq 5$  f.p.s.

DEPTH OF SUBCUT TO REMOVE UNSUITABLE MATERIAL  
SHALL NOT EXCEED 2 FEET BELOW BOTTOM OF FOOTING.

**Figure 12.3.3.1**  
**Design 1 Scour Protection for Arch or 3-Sided Bridge**



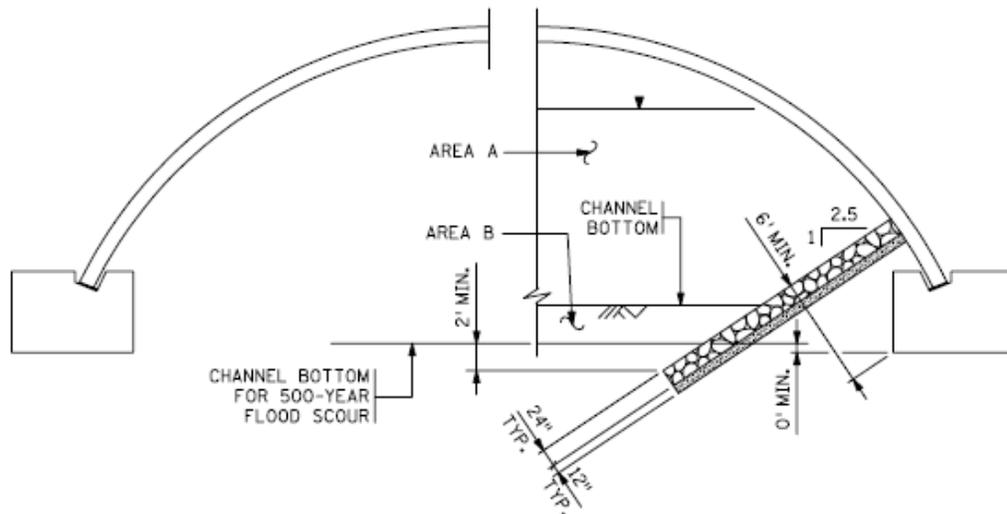
DESIGN 2A SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

24" CLASS IV RIPRAP  
 12" GRANULAR FILTER

AVERAGE VELOCITY THROUGH STRUCTURE:

$V_{100} \leq 5.5$  f.p.s.  
 $V_{500} \leq 6.5$  f.p.s.

DEPTH OF SUBCUT TO REMOVE UNSUITABLE MATERIAL  
 SHALL NOT EXCEED 2 FEET BELOW BOTTOM OF FOOTING.



DESIGN 2B SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

24" CLASS IV RIPRAP  
 12" GRANULAR FILTER

AVERAGE VELOCITY THROUGH STRUCTURE:

$V_{100} \leq 5.5$  f.p.s.  
 $V_{500} \leq 6.5$  f.p.s.

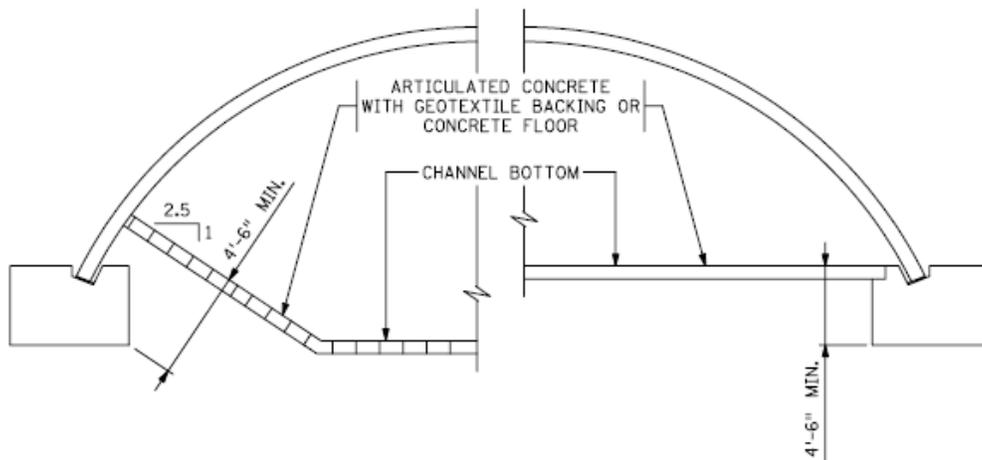
DEPTH OF SUBCUT TO REMOVE UNSUITABLE MATERIAL  
 SHALL NOT EXCEED 2 FEET BELOW BOTTOM OF FOOTING.

AREA A = AREA BOUNDED BY CHANNEL BOTTOM AND WATER SURFACE.

AREA B = AREA BOUNDED BY CHANNEL BOTTOM AND 500-YEAR FLOOD SCOURED CHANNEL BOTTOM.

**Figure 12.3.3.2**

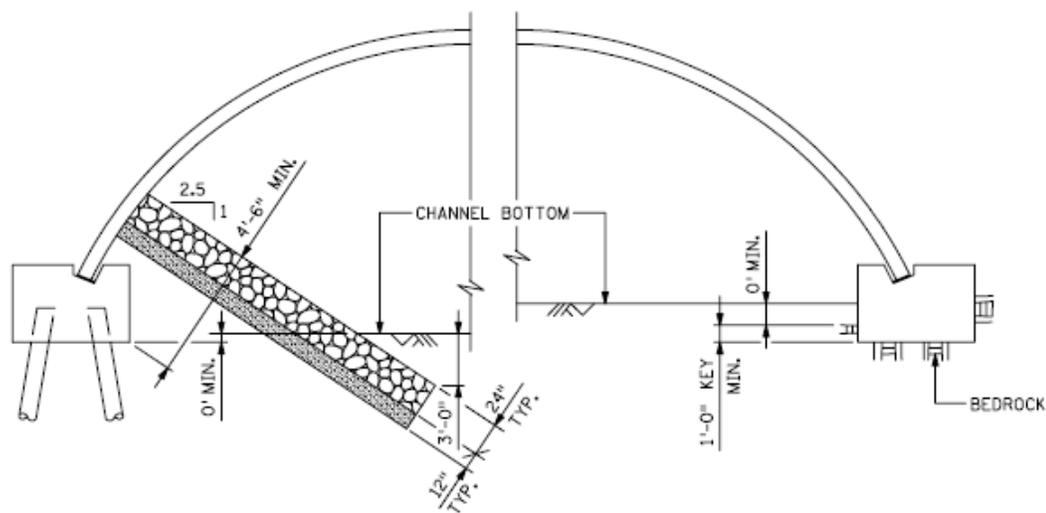
**Design 2A and 2B Scour Protection for Arch or 3-Sided Bridge**



DESIGN 3 SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

- A. CONCRETE FLOOR OR
  - B. ARTICULATED CONCRETE WITH GEOTEXTILE BACKING
- AVERAGE VELOCITY THROUGH STRUCTURE:  
 $V_{100} \leq 7.5$  f.p.s.

**Figure 12.3.3.3**  
**Design 3 Scour Protection for Arch or 3-Sided Bridge**



DESIGN 4 SCOUR PROTECTION FOR ARCH OR 3-SIDED BRIDGE

- A. FOOTINGS ON PILING OR
  - B. SPREAD FOOTING KEYED INTO HARD BEDROCK
- AVERAGE VELOCITY THROUGH STRUCTURE:  
 $V_{100} > 7.5$  f.p.s.

NOTE: IT IS THE CONTRACTOR'S RESPONSIBILITY TO PROTECT THE FOOTING FOUNDATION FROM SCOUR UNTIL THE RIPRAP HAS BEEN PLACED.

**Figure 12.3.3.4**  
**Design 4 Scour Protection for Arch or 3-Sided Bridge**

**12.4 Use of  
Long-Span  
Corrugated  
Steel Structures**

The design requirements for long span corrugated steel structures are currently being updated. At a minimum, corrugated steel structures shall meet the criteria below. Contact the State Bridge Design Engineer for approval before utilizing on a project.

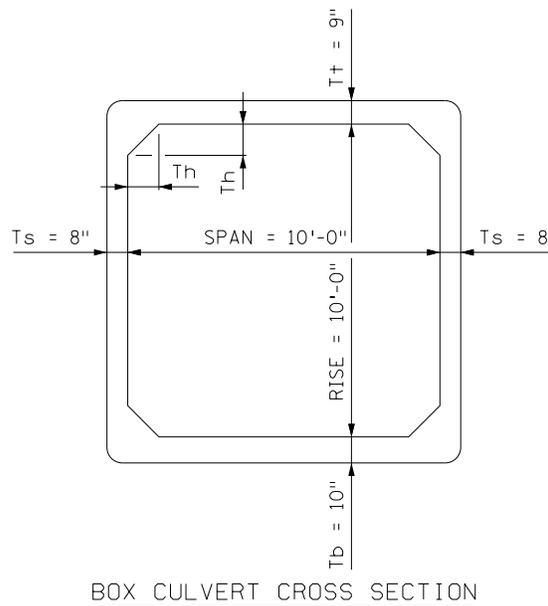
1. These structures are not permitted for use as vehicle underpasses.
2. All designs utilizing Federal, State, or State Aid funds will be reviewed by MnDOT. The review will verify compliance with AASHTO Specifications, MnDOT Specifications, MnDOT detail sheets, and MnDOT design guidelines.
3. Standard Plan Sheets developed by MnDOT indicate the span, rise, invert elevation, profile grade over, and hydraulic characteristics of the structure.
4. These structures are considered arch bridges that depend structurally on the interaction of the structural plate liner and good quality soil, which is carefully compacted. Balanced placement of backfill and close field supervision of backfilling operations is required.
5. Detailed plans that include structural computations and special provisions shall be certified by a qualified professional engineer registered in the State of Minnesota.
6. MnDOT Projects
  - a. When MnDOT proposes the use of these structures, the Bridge Office will determine the shape, invert elevation, roadway and hydraulic data, and complete a design plan with special provisions to be forwarded to the District Engineer.
  - b. A copy of the design plan and special provisions shall be forwarded to the Bridge Standards Engineer.
7. State Aid Projects
  - a. For State Aid approval, the county or municipal engineer shall submit 3 copies of an engineering report on the structure's hydraulic characteristics, special provisions, a design detail plan, and funding request forms to the MnDOT State Aid Office.
  - b. Final funding approval by the State Aid Office to include approval of the design plan and hydraulic characteristics by the Bridge Office.
8. Scour protection for long span corrugated steel structures shall be the same as those for precast concrete arch structures (see Section 12.3.3).
9. The Foundation Engineer will determine the suitability of the foundation material, and provide recommendations regarding required sub-cuts.

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**12.5 10'x10'  
Precast Concrete  
Box Culvert  
Design Example**

This example illustrates the design of a single barrel precast concrete box culvert. After determining the load components and design load combinations, the design of the flexural reinforcement is presented. The example concludes with a shear check and an axial load capacity check.

Inside dimensions of the box culvert (Span x Rise) are 10'-0" by 10'-0" with 12" haunches (Th). The fill height (H) above the culvert is 6'-0". A typical section of the culvert is shown in Figure 12.5.1. Material and design parameters are given in Table 12.5.1.



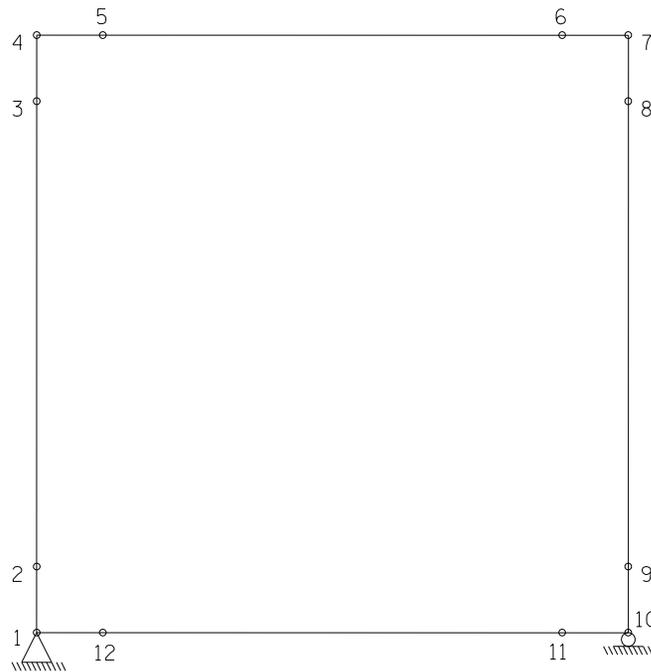
**Figure 12.5.1**

**Table 12.5.1  
Material and Design Parameters**

Unit Weights	Reinforced Concrete, $\gamma_c$	0.150 kcf
	Water, $\gamma_w$	0.0624 kcf
	Soil, $\gamma_s$	0.120 kcf
Concrete	Compressive Strength, $f'_c$	5.0 ksi
	Top Slab Thickness, $T_t$	0.75 ft
	Bottom Slab Thickness, $T_b$	0.83 ft
	Wall Thickness, $T_s$	0.67 ft
	Haunch Thickness, $T_h$	12 in
	Reinforcement Clear Cover	2 in
Steel Reinforcement	Modulus of Elasticity, $E_s$	29,000 ksi
	Yield Strength, $f_y$	65 ksi
	Maximum Wire Size	W23
	Maximum Wire Spacing	4 in transverse 8 in longitudinal

The approximate strip method is used for the design with the 1'-0" wide design strip oriented parallel to the direction of traffic.

A 2-Dimensional (2D) plane frame model is used to analyze the box culvert. Beam elements in the 2D model are assumed to be centered in the concrete members. The model is assumed to be externally supported by a pinned support on one end and a roller support on the other end. In addition, the model is always assumed to be in equilibrium so external reactions to loads applied to the structure were assumed to act equal and opposite. A "w" dimension of 1 ft is added to the calculations to convert the units to klf for consistency with national conventions.



**Figure 12.5.2**  
**2D Plane Frame Model**

**A. Dead Load**

The self-weight of the culvert top slab is:

$$DC_{top} = Tt \cdot w \cdot \gamma_c = 0.75 \cdot 1 \cdot 0.150 = 0.113 \text{ klf}$$

The total self-weight of the culvert top slab is:

$$DC_{top} = Tt \cdot w \cdot \gamma_c \cdot (\text{Span} + Ts) = 0.75 \cdot 1 \cdot 0.150 \cdot (10 + 0.67) = 1.20 \text{ kips}$$

The self-weight of one culvert side wall is:

$$DC_{side} = Ts \cdot w \cdot \gamma_c \cdot \left( \text{Rise} + \frac{Tt}{2} + \frac{Tb}{2} \right) = 0.67 \cdot 1 \cdot 0.150 \cdot \left( 10 + \frac{0.75}{2} + \frac{0.83}{2} \right) = 1.08 \text{ kips}$$

The self-weight of one haunch is:

$$DC_{\text{haunch}} = 0.5 \cdot Th \cdot w \cdot Th \cdot \gamma_c = 0.5 \cdot 1 \cdot 1 \cdot 1 \cdot 0.150 = 0.075 \text{ kips}$$

The top slab weight, wall weights, and all four haunch weights are applied to the bottom slab as an upward reaction from the soil assuming an equivalent uniform pressure. The bottom slab weight is not applied in the model because its load is assumed to be directly resisted by the soil.

$$\begin{aligned} DC_{\text{bottom}} &= (DC_{\text{top}} + 4DC_{\text{haunch}} + 2DC_{\text{side}}) \cdot \left( \frac{1}{\text{Span} + Ts} \right) \\ &= (1.20 + 4 \cdot 0.075 + 2 \cdot 1.08) \cdot \left( \frac{1}{10 + 0.67} \right) = 0.343 \text{ klf} \end{aligned}$$

### **B. Earth Pressure Loads**

#### **[12.11.2]**

The weight of fill on top of the culvert produces vertical earth pressure (EV). The fill height is measured from the top surface of the top slab to the top of the pavement or fill. Per Table 12.5.1, the unit weight of the fill is 0.120 kcf.

The interaction factor for embankment conditions is dependent on the height of fill (H) and the outside width of the culvert ( $B_c$ ):

#### **[12.11.2.2.1-2]**

$$F_e = 1 + 0.20 \cdot \left( \frac{H}{B_c} \right) = 1 + 0.20 \cdot \left( \frac{6}{2 \cdot 0.67 + 10} \right) = 1.11$$

The design vertical earth pressure at the top of the culvert is:

#### **[12.11.2.2.1-1]**

$$EV = F_e \cdot \gamma_s \cdot H \cdot w = 1.11 \cdot 0.120 \cdot 6 \cdot 1 = 0.799 \text{ klf}$$

#### **[3.11.5]**

#### **[3.11.7]**

The lateral earth pressure (EH) on the culvert is found using the equivalent fluid method. For at-rest conditions, a maximum equivalent fluid unit weight of 0.060 kcf and a minimum equivalent fluid unit weight of 0.030 kcf are used.

At the top of the culvert, the lateral earth pressure is:

$$EH_{\text{max}} = \gamma_{\text{max}} \cdot H \cdot w = 0.060 \cdot 6 \cdot 1 = 0.360 \text{ klf}$$

$$EH_{\text{min}} = \gamma_{\text{min}} \cdot H \cdot w = 0.030 \cdot 6 \cdot 1 = 0.180 \text{ klf}$$

At the bottom of the culvert, the lateral earth pressure is is:

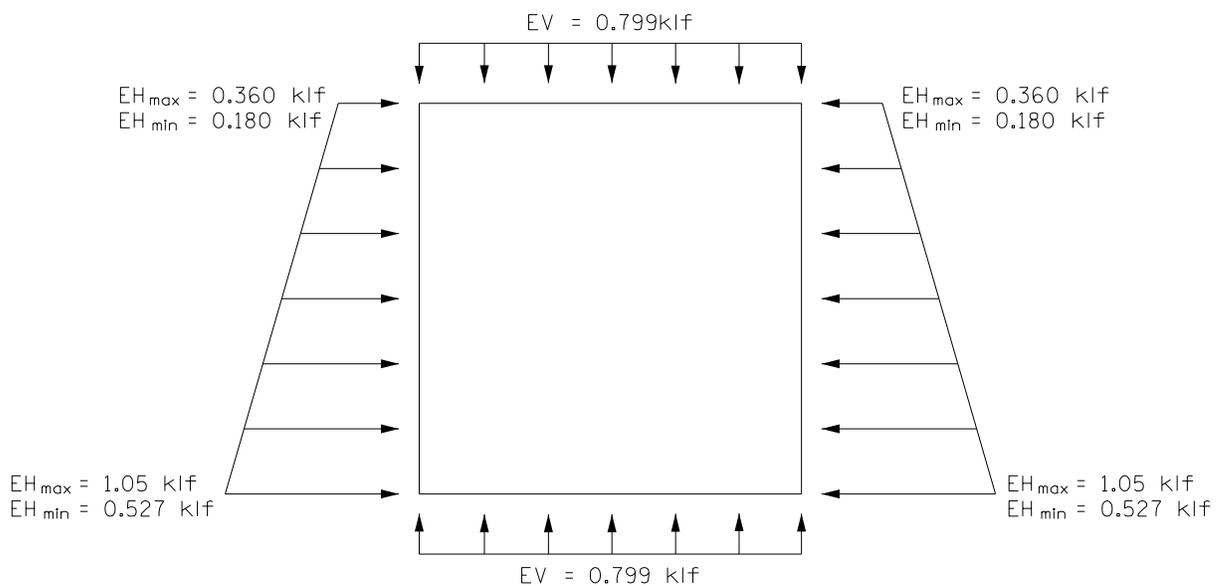
$$EH_{max} = \gamma_{max} \cdot (H + Tt + Rise + Tb) \cdot w$$

$$= 0.060 \cdot (6 + 0.75 + 10 + 0.83) \cdot 1 = 1.05 \text{ klf}$$

$$EH_{min} = \gamma_{min} \cdot (H + Tt + Rise + Tb) \cdot w$$

$$= 0.030 \cdot (6 + 0.75 + 10 + 0.83) \cdot 1 = 0.527 \text{ klf}$$

Figure 12.5.3 illustrates the vertical and lateral earth pressures applied to the box culvert.



**Figure 12.5.3**  
**Earth Loads**

**C. Live Load  
Surcharge  
[3.11.6.4]**

Use an active coefficient of lateral earth pressure  $k_a$  equal to 0.33.

The height for the live load surcharge calculation at the top of the culvert is the distance from the top surface of the top slab to the top of the pavement or fill.

The height is:

$$H_{top \text{ of culvert}} = H_1 = 6 \text{ ft}$$

The equivalent fill height,  $h_{eq}$  is dependent on the depth of fill and can be found using AASHTO Table 3.11.6.4-1.

By interpolation, the equivalent height for a fill depth of 6 ft is:

$$h_{eq1} = 4 - \left( \frac{6 - 5}{10 - 5} \right) (4 - 3) = 3.80 \text{ ft}$$

The corresponding lateral live load surcharge on the top of the culvert is given as:

$$LS_{top} = k_a \cdot \gamma_s \cdot h_{eq} \cdot w = 0.33 \cdot 0.120 \cdot 3.80 \cdot 1 = 0.150 \text{ klf}$$

The height for the live load surcharge calculation at the bottom of the culvert is the distance from the bottom surface of the bottom slab to the top of the pavement or fill.

$$H_2 = H + T_t + \text{Rise} + T_b = 6 + 0.75 + 10 + 0.83 = 17.58 \text{ ft}$$

Again using interpolation and AASHTO Table 3.11.6.4.1, the equivalent height is:

$$h_{eq2} = 3 - \left( \frac{17.58 - 10}{20 - 10} \right) (3 - 2) = 2.24 \text{ ft}$$

The lateral live load surcharge located at the bottom of the culvert is given as:

$$LS_{bottom} = k_a \cdot \gamma_s \cdot h_{eq} \cdot w = 0.33 \cdot 0.120 \cdot 2.24 \cdot 1 = 0.089 \text{ klf}$$

**D. Water Load**  
**[3.7.1]**

Designers need to consider load cases where the culvert is full of water as well as cases where the culvert is empty. A simple hydrostatic distribution is used for the water load:

At the inside of the culvert, the lateral water pressure is:

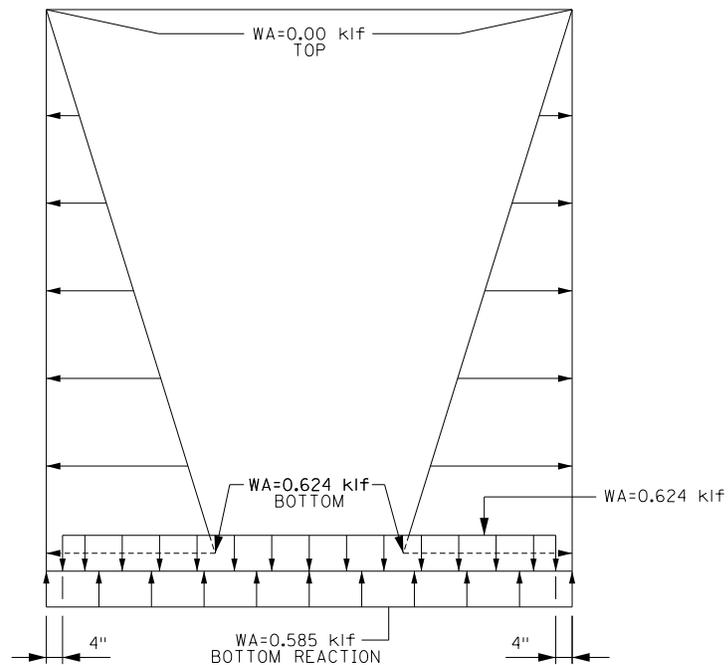
$$WA_{top} = 0.00 \text{ klf}$$

$$WA_{bottom} = \gamma_w \cdot \text{Rise} \cdot w = 0.0624 \cdot 10 \cdot 1 = 0.624 \text{ klf}$$

Using a 2D frame model there is an opposite upward reaction from the soil caused by the water inside the culvert:

$$WA_{bottom \text{ reaction}} = \frac{WA_{bottom} \cdot \text{Span}}{(\text{Span} + T_s)} = \frac{0.624 \cdot 10}{(10 + 0.67)} = 0.585 \text{ klf}$$

The water load is illustrated in Figure 12.5.4.



**Figure 12.5.4**  
**Water Load**

**E. Live Load**  
**[3.6.1.3.3]**

The design live loads include the HL-93 truck and tandem loads. Since the span of the box culvert is less than 15 ft, no lane load is applied.

**[3.6.2.2]**

**Dynamic Load Allowance**

The dynamic load allowance (IM) for culverts and other buried structures is reduced based on the depth of fill over the culvert. For strength and service limit states:

**[3.6.2.2-1]**

$$IM = 33 \cdot (1.0 - 0.125 \cdot D_E) = 33 \cdot (1.0 - 0.125 \cdot 6.0) = 8.3\%$$

The dynamic load allowance may not be taken less than zero.

**[3.6.1.2.6]**

**Live Load Distribution**

Live loads are assumed to distribute laterally with depth. The specifications permit designers to increase the footprint of the load with increasing depth of fill. The load is assumed to spread laterally 1.15 times H horizontally in each direction for every foot of fill above the culvert. The intensity of live loads at any depth is assumed to be uniform over the entire footprint.

**[3.6.1.2.5]**

The assumed tire contact area for each wheel has a width of 20 inches and a length of 10 inches.

Using the distances between wheel lines and axles, the live load intensities at the top of the box culvert can be found. For truck and tandem loadings, the influence area or footprint of the live load is found first. Then the sum of the weights of the wheels is used to determine the intensity of the live load.

**[3.6.1.1.2]**

To determine the live load, use multiple presence factors (MPF). A single loaded lane with a MPF of 1.20 is used for strength and service limit states.

A single HL-93 truck axle configuration produces a live load intensity of:

$$W_{LL+IM} = \frac{2 \cdot P_w \cdot MPF \cdot (1+IM)}{W \cdot L} = \frac{2 \cdot 16 \cdot 1.20 \cdot (1+0.083)}{14.57 \cdot 7.73} = 0.369 \text{ klf}$$

where:

$$W = \text{Axle}_{\text{spacing}} + W_{\text{tire}} + 1.15 \cdot H = 6 + 1.67 + 1.15 \cdot 6 = 14.57 \text{ ft}$$

$$L = L_{\text{tire}} + 1.15 \cdot H = 0.83 + 1.15 \cdot 6 = 7.73 \text{ ft}$$

A tandem truck axle configuration produces a live load intensity of:

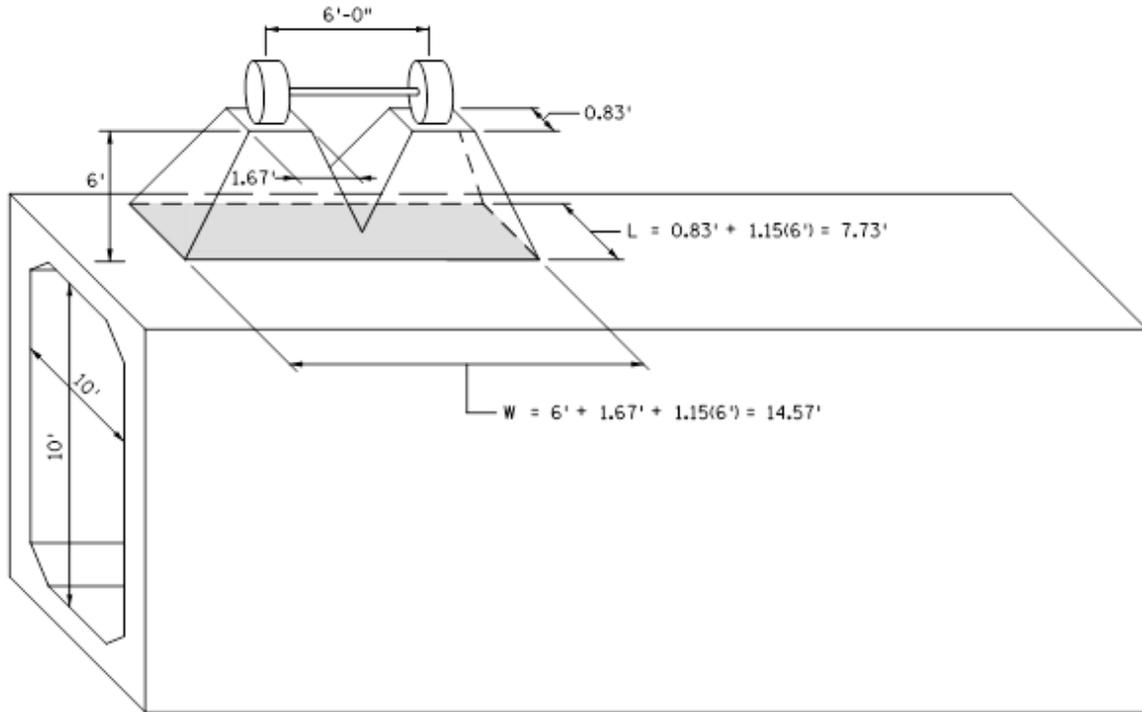
$$W_{LL+IM} = \frac{4 \cdot P_w \cdot MPF \cdot (1+IM)}{W \cdot L} = \frac{4 \cdot 12.5 \cdot 1.20 \cdot (1+0.083)}{14.57 \cdot 11.73} = 0.380 \text{ klf}$$

where:

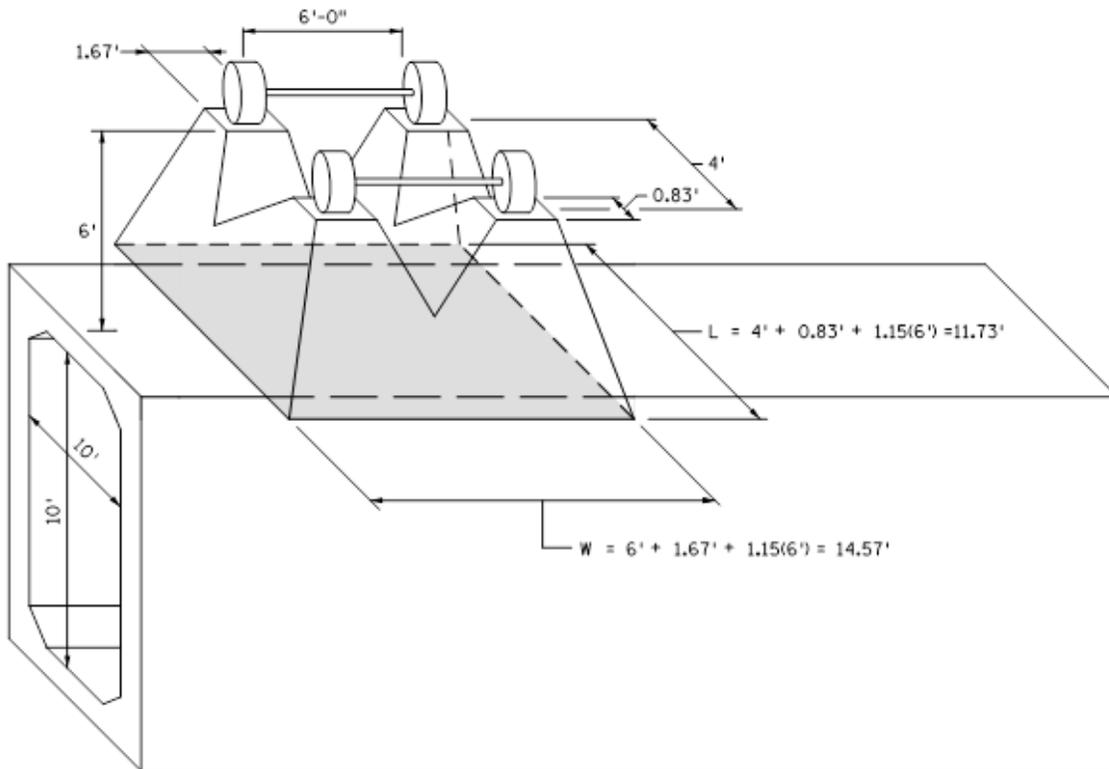
$$W = \text{as previously defined}$$

$$L = \text{Axle Spacing} + L_{\text{tire}} + 1.15 \cdot H = 4 + 0.83 + 1.15 \cdot 6 = 11.73 \text{ ft}$$

The live load intensities of the single and tandem axle configurations are compared. Since the tandem axle configuration produces a live load intensity slightly larger than that of the single axle configuration, the tandem axle configuration is used for design in both the strength and service limit states. Figure 12.5.5 illustrates the different live loads.



HL-93 TRUCK



HL-93 TANDEM

**Figure 12.5.5**  
**HL-93 Truck and Tandem Live Load Distribution**

**F. Select  
Applicable  
Load  
Combinations,  
Load Factors,  
and Load  
Modifiers**

**Strength Limit State:**

- Ia. Maximum vertical load and maximum horizontal load:  
 $1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + (1.35)(1.05)EH_{max} + 1.75LS$
- Ib. Maximum vertical load and minimum horizontal load:  
 $1.25DC + (1.30)(1.05)EV + 1.75(LL+IM) + 1.00WA + (0.9/1.05)EH_{min}$
- Ic. Minimum vertical load and maximum horizontal load:  
 $0.90DC + (0.90/1.05)EV + (1.35)(1.05)EH_{max} + 1.75LS$

**Service Limit State:**

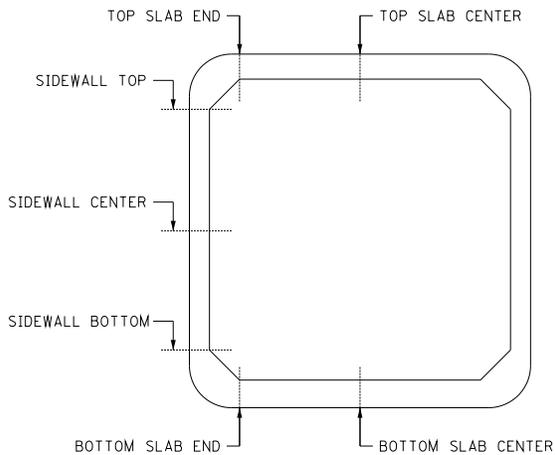
- Ia. Maximum vertical load and maximum horizontal load:  
 $1.00DC + 1.00EV + 1.00(LL+IM) + 1.00EH_{max} + 1.00LS$
- Ib. Maximum vertical load and minimum horizontal load:  
 $1.00DC + 1.00EV + 1.0(LL+IM) + 1.00WA + 1.00EH_{min}$
- Ic. Minimum vertical load and maximum horizontal load:  
 $1.00DC + 1.00EV + 1.00EH_{max} + 1.00LS$

**G. Summary of  
Analysis Results**

A structural analysis is performed using a standard commercial matrix-analysis program. The bottom slab of the box culvert is assumed rigid compared to the subgrade. Reactions to vertical loads applied to the culvert (earth, water, live load) are assumed to be carried by uniform, triangular or trapezoidal distributed reactions applied to the bottom slab. Box culverts supported on stiff or rigid subgrades (rock) would require further investigation. The haunches are included in the analysis by increasing the thickness of members near each corner.

The internal forces at several locations of the box are presented in Tables 12.5.2 through 12.5.6. The sign convention for moment in the tables is: positive moment causes tension on the inside face of the culvert and negative moment causes tension on the outside face. The sign convention for thrust is: positive represents compression. The moments and thrust presented at top, bottom, or end locations are at the location where the typical section and haunch meet (Figure 12.5.6). The shear forces presented in Tables 12.5.4 and 12.5.5 are at the critical shear location, which is taken as the effective depth for shear ( $d_v$ ) beyond the haunch to typical section intersection. The shear forces presented are the "governing" shear forces which are the shear with corresponding moments that give the lowest capacity/design (c/d) ratios.

**[C5.13.3.6.1]**



**Figure 12.5.6**  
**Structural Analysis Locations**

**Table 12.5.2**

**Structural Analysis Results: Moments (unfactored, kip-in)**

	DC	EV	EH <sub>max</sub>	EH <sub>min</sub>	LS	WA	LL+IM (Pos)	LL+IM (Neg)
Sidewall Top	-4.31	-44.97	-4.84	-2.42	0.36	4.13	-	-21.57
Sidewall Center	-11.06	-39.44	63.02	31.51	10.7	-26.77	-	-18.82
Sidewall Bottom	-17.73	-33.97	-4.80	-2.40	-1.93	2.32	-	-16.21
Top Slab Center	17.19	89.03	-50.54	-25.27	-9.30	19.93	42.49	-
Top Slab End	6.39	12.60	-50.54	-25.27	-9.30	19.93	7.93	-2.35
Bottom Slab Center	38.43	103.87	-70.62	-35.31	-11.09	28.7	49.57	-
Bottom Slab End	5.50	27.43	-70.62	-35.31	-11.09	32.45	14.47	-

**Table 12.5.3**

**Moment Load Combinations (kip-in)**

	Strength			Service		
	Ia	Ib	Ic	Ia	Ib	Ic
Sidewall Top	-110.74	-102.45	-48.66	-75.33	-69.13	-53.76
Sidewall Center	40.39	-100.36	64.29	23.22	-64.58	23.22
Sidewall Bottom	-107.08	-96.63	-55.26	-74.64	-67.99	-58.43
Top Slab Center	129.47	215.66	3.87	88.88	143.39	46.39
Top Slab End	-66.85	37.33	-71.37	-43.20	21.58	-40.85
Bottom Slab Center	157.05	275.01	4.09	110.16	185.27	60.58
Bottom Slab End	-75.20	71.83	-91.06	-48.78	44.54	-48.78

**Table 12.5.4**  
**Structural Analysis Results: Shear (unfactored, kips)**

	DC	EV	EH <sub>max</sub>	EH <sub>min</sub>	LS	WA	LL+IM (Pos)	LL+IM (Neg)
Sidewall Top*	0.14	-0.11	-2.26	-1.13	-0.42	0.9	0.01	-0.09
Sidewall Center	0.14	-0.11	-0.28	-0.14	0.02	0.23	0.01	-0.09
Sidewall Bottom*	0.14	-0.11	2.74	1.37	0.43	-1.31	0.01	-0.09
Top Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.41	-0.4
Top Slab End*	-0.39	-2.75	0.00	0.00	0.00	0.00	-	-1.35
Bottom Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.26	-0.27
Bottom Slab End*	1.16	2.69	0.00	0.00	0.00	-0.13	1.29	0.29

\*Shear given at  $d_v$  away from haunch

**Table 12.5.5**  
**Governing Shear Load Combinations (kips)**

	Strength			Service		
	Ia	Ib	Ic	Ia	Ib	Ic
Sidewall Top*	-4.10	-0.22	-3.93	-2.76	-0.30	-2.66
Sidewall Center	-0.31	0.15	-0.32	-0.22	0.13	-0.22
Sidewall Bottom*	4.48	-0.28	4.65	3.09	-0.01	3.19
Top Slab Center	0.72	0.72	0.00	0.41	0.41	0.00
Top Slab End*	-6.61	-6.61	-2.71	-4.50	-4.50	-3.14
Bottom Slab Center	-0.47	-0.47	0.00	-0.27	-0.27	0.00
Bottom Slab End*	7.36	7.23	3.35	5.13	5.0	3.84

\*Shear given at  $d_v$  away from haunch

**Table 12.5.6**  
**Axial Thrust Load Combinations (kips)**

	Strength			Service		
	Ia	Ib	Ic	Ia	Ib	Ic
Sidewall Top	11.43	11.43	4.96	7.87	7.87	5.73
Sidewall Center	11.44	11.44	4.96	7.88	7.88	5.73
Sidewall Bottom	11.44	11.44	4.96	7.88	7.88	5.73
Top Slab Center	5.88	0.47	5.77	3.95	0.65	3.89
Top Slab End	5.85	0.45	5.77	3.93	0.64	3.89
Bottom Slab Center	8.26	-0.11	8.29	5.69	0.38	5.70
Bottom Slab End	8.26	-0.11	8.29	5.69	0.38	5.70

The values in Tables 12.5.2 through 12.5.6 include dynamic load allowance and multiple presence factors.

**H. Investigate**  
**Strength Limit**  
**State for Flexure**  
**[5.7.2.2]**  
**[5.7.3.2]**  
**[12.5.5]**

Determine the required area of flexural reinforcement to satisfy the Strength I load combinations.

The resistance factor,  $\phi$ , for flexure is 1.0 for precast box culverts.

$$M_u = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$$

The depth of the compression block is:

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

Substituting for "a" in the first equation:

$$M_u = \phi \cdot A_s \cdot f_y \cdot \left[ d - \frac{A_s \cdot f_y}{1.7 \cdot f'_c \cdot b} \right]$$

Inserting values for  $f_y$ ,  $b$ , and  $\phi$ :

$$M_u = 1.0 \cdot A_s \cdot 65 \cdot \left[ d - \frac{A_s \cdot 65}{1.7 \cdot 5 \cdot 12} \right] \cdot \left[ \frac{1}{12} \right]$$

Manipulate to get a quadratic equation:

$$3.45 \cdot A_s^2 - 5.42 \cdot A_s \cdot d + M_u = 0$$

$$A_s = \frac{5.42 \cdot d - \sqrt{29.34 \cdot d^2 - 13.81 \cdot M_u}}{6.91}$$

**Sidewall:**

Size the reinforcement assuming "d" dimensions based on an average 1 inch diameter wire, ( $d_w = 1.00$  in) and a clear cover of 2 in.

$$d = \text{thickness} - \text{cover} - \frac{d_w}{2} = 8 - 2 - \frac{1}{2} = 5.50 \text{ in}$$

Referring to Table 12.5.2, the peak moment for tension on the outside face is 110.74 k-in (top, Strength Ia). Insert  $d$  and  $M_u$  values to compute  $A_s$ . The required area of steel is 0.321 in<sup>2</sup>/ft. For conservatism round up to 0.33 in<sup>2</sup>/ft.

The peak moment for tension on the inside face is 64.29 k-in (center, Strength Ic). The required area of steel is 0.19 in<sup>2</sup>/ft.

**Top Slab:**

For the top slab "d" is:

$$d = 9 - 2 - \frac{1}{2} = 6.50 \text{ in}$$

The peak moment for tension on the outside face is 71.37 k-in (Strength Ic). The required area of steel is 0.18 in<sup>2</sup>/ft.

The peak moment for tension on the inside face is 215.66 k-in (Strength Ib). The required area of steel is 0.54 in<sup>2</sup>/ft.

**Bottom Slab:**

$$d = 10 - 2 - \frac{1}{2} = 7.50 \text{ in}$$

The peak moment for tension on the outside face is 91.06 k-in. The required area of steel is 0.19 in<sup>2</sup>/ft.

The peak moment for tension on the inside face is 275.01 k-in. The required area of steel is 0.60 in<sup>2</sup>/ft.

**I. Check Crack  
Control  
[5.7.3.4]**

**[C5.7.3.4]**

To ensure that the primary reinforcement is well distributed, crack control equations are checked. The equations are dependent on the tensile stress in steel reinforcement at the service limit state, the concrete cover, and the geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level ( $\beta_s$ ). The exposure factor,  $\gamma_e$ , is 0.75, since culverts are substructures exposed to water (Class 2).

The wire spacing,  $s$ , must satisfy:

**[5.7.3.4-1]**

$$s \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

Solve the equation above for the reinforcement stress at service,  $f_{ss}$ :

$$f_{ss} \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot (s + 2 \cdot d_c)} \leq 0.6f_y$$

The strain ratio,  $\beta_s$ , is defined as:

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$$

**Top Slab:**

For the top slab inside face, the governing service limit state moment is 143.39 k-in. The axial thrust is 0.65 kips and is accounted for in the crack control check per AASHTO C.12.11.3-1. Spacing of the wires is assumed to be 4 inches and the area of flexural reinforcement is 0.54 in<sup>2</sup>/ft.

$$d_c = \text{Cover} + \frac{1}{2}d_w = 2 + \frac{1}{2} \cdot 1 = 2.50\text{in}$$

Then solve for  $\beta_s$ :

$$\beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1 + \frac{2.50}{0.7 \cdot (9 - 2.50)} = 1.55$$

The allowable reinforcement stress,  $f_{ss}$  can then be calculated as:

$$f_{ss} = \frac{700 \cdot \gamma_e}{\beta_s \cdot (s + 2d_c)} = \frac{700 \cdot 0.75}{1.55 \cdot (4 + 2 \cdot 2.50)} = 37.63 \text{ ksi}$$

$$0.6f_y = 0.6 \cdot 65 = 39.00 \text{ ksi} > 37.63 \text{ ksi} \quad \text{Use } 37.63 \text{ ksi}$$

**[C12.11.3]**

Find the actual stress provided in the steel:

$$e = \frac{M_s}{N_s} + d - \frac{h}{2} = \frac{143.39}{0.65} + 6.5 - \frac{9}{2} = 222.60$$

$$j = 0.74 + 0.1 \cdot \left(\frac{e}{d}\right) = 0.74 + 0.1 \cdot \left(\frac{222.60}{6.5}\right) = 4.16$$

For "j" use the smaller of 4.16 or 0.9, then  $j=0.9$

$$i = \frac{1}{1 - \frac{j \cdot d}{e}} = \frac{1}{1 - \frac{0.9 \cdot 6.5}{222.60}} = 1.027$$

**[C12.11.3-1]**

$$f_s = \frac{M_s + N_s \cdot \left(d - \frac{h}{2}\right)}{A_s \cdot j \cdot i \cdot d} = \frac{143.39 + 0.65 \cdot \left(6.5 - \frac{9}{2}\right)}{0.54 \cdot 0.9 \cdot 1.027 \cdot 6.5}$$

$$= 44.60 \text{ ksi} > 37.63$$

No Good

Increase the area of steel provided, so that  $f_s$  is less than  $f_{ss}$ . The new area of steel is given as:

$$A_{\text{scrack}} = \frac{f_s}{f_{ss}} \cdot A_s = \frac{44.60}{37.63} \cdot 0.54 = 0.64 \text{ in}^2/\text{ft}$$

For the top slab outside face crack control did not govern. See Table 12.5.6 for results.

**Bottom Slab:**

The area of steel for the bottom slab inside face is evaluated with a service moment of 185.27 k-in, an axial thrust of 0.38 kips, and  $d_c$  equal to 2.50 inches. The required area of steel to satisfy crack control for the bottom slab inside face is 0.70 in<sup>2</sup>/ft.

**Sidewall:**

The area of steel for the sidewall inside face is evaluated with a service moment of 23.22 k-in, an axial thrust of 5.73 kips, and  $d_c$  equal to 2.50 inches. The required area of steel to satisfy crack control for the sidewall inside face is 0.03 in<sup>2</sup>/ft.

**J. Check Fatigue  
[C12.5.3]**

Fatigue check calculations are not required for the design of box culverts.

**K. Check Minimum  
Reinforcement  
[12.11.4.3.2]**

For precast culverts, the minimum amount of flexural reinforcement in the cross section is a percentage of the gross area:

Minimum sidewall flexural reinforcement:

$$A_s = 0.002 \cdot b \cdot T_t = 0.002 \cdot 8 \cdot 12 = 0.20 \text{ in}^2/\text{ft}$$

Minimum top slab flexural reinforcement:

$$A_s = 0.002 \cdot b \cdot T_s = 0.002 \cdot 9 \cdot 12 = 0.22 \text{ in}^2/\text{ft}$$

Minimum bottom slab reinforcement:

$$A_s = 0.002 \cdot b \cdot T_b = 0.002 \cdot 10 \cdot 12 = 0.24 \text{ in}^2/\text{ft}$$

For precast concrete box culverts, the MnDOT minimum reinforcement requirement is 0.06 in<sup>2</sup>/ft, regardless of the size of the box culvert.

**L. Check  
Maximum  
Reinforcement  
Limit  
[5.5.4.2]  
[5.7.2.1]**

The strain in the reinforcement is checked to ensure that the section is tension controlled. For a resistance factor of 1.0 to be used for flexure, the reinforcement strain must be at least 0.005.

This is satisfied if:

$$\frac{c}{d} < 0.375$$

where:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b}$$

$$f'_c = 5 \text{ ksi}$$

$$0.65 \leq \beta_1 = 0.85 - 0.05 \cdot (f'_c - 4.0) = 0.80 \leq 0.85$$

**Sidewall:**

Outside face	$c = 0.513 \text{ in}$	$\frac{c}{d} = \frac{0.513}{5.5} = 0.09$	<u>OK</u>
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**Top Slab:**

Inside face	$c = 1.02 \text{ in}$	$\frac{c}{d} = \frac{1.02}{6.5} = 0.16$	<u>OK</u>
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**Bottom Slab:**

Inside face	$c = 1.11 \text{ in}$	$\frac{c}{d} = \frac{1.11}{7.5} = 0.15$	<u>OK</u>
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Minnesota Concrete Pipe Association (MCPA) members also prefer to have a maximum reinforcement ratio of  $0.6\rho_b$  to limit congestion during fabrication. The balanced reinforcement ratio is given by:

$$\rho_b = \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left[ \frac{87}{87 + f_y} \right] = \frac{0.85 \cdot 0.80 \cdot 5}{65} \cdot \left[ \frac{87}{87 + 65} \right] = 0.0299$$

$$\rho \leq 0.60 \cdot \rho_b = 0.60 \cdot 0.0299 = 0.018$$

For the top slab  $b=12 \text{ in.}$ ,  $d= 6.5 \text{ in.}$ , and  $A_s = 0.64 \text{ in}^2$ , the member reinforcement ratio is given as:

$$\rho = \frac{A_s}{A_c} = \frac{A_s}{b \cdot d} = \frac{0.64}{12 \cdot 6.5} = 0.0082 < 0.018 \quad \text{OK}$$

**Sidewall:**

For the sidewall with  $b=12 \text{ in.}$ ,  $d= 5.5 \text{ in.}$ , and  $A_s = 0.20 \text{ in}^2$  the reinforcement ratio is  $0.0030 < 0.018$ . OK

**Bottom Slab:**

For the bottom slab with  $b=12 \text{ in.}$ ,  $d= 7.5 \text{ in.}$ , and  $A_s = 0.70 \text{ in}^2$  the reinforcement ratio is  $0.0077 < 0.018$ . OK

**Table 12.5.7**  
**Flexural Design Calculation Summary**

		Sidewall		Top Slab		Bottom Slab	
		Inside	Outside	Inside	Outside*	Inside	Outside*
Strength	Moment (k-in)	64.29	110.74	215.66	-	275.01	-
	Assumed d (in)	5.5	5.5	6.5	-	7.5	-
	Req'd steel area (in <sup>2</sup> /ft)	0.19	<b>0.33</b>	0.54	-	0.60	-
Service	Moment (k-in)	23.22	75.33	143.39	-	185.27	-
	Axial Thrust (kip)	5.73	7.87	0.65	-	0.38	-
	Assumed d (in)	5.5	5.5	6.5	-	7.5	-
	Assumed d <sub>c</sub> (in)	2.5	2.5	2.5	-	2.5	-
	f <sub>ss</sub> (ksi)	35.37	35.37	37.64	-	39.00	-
	f <sub>s</sub> (ksi)	9.13	29.49	37.64	-	39.00	-
	Req'd steel area for crack control (in <sup>2</sup> /ft)	0.03	0.28	<b>0.64</b>	-	<b>0.70</b>	-
Min Check	0.002 A <sub>g</sub> (in <sup>2</sup> /ft)	<b>0.20</b>	0.20	0.22	<b>0.22</b>	0.24	<b>0.24</b>

\*The minimum reinforcement always governs (given MnDOT's reinforcement lap criteria).

**M. Summary of Required Flexural Reinforcement**

The final amount of reinforcement is:

**Sidewall:**

Outside face      As1 = 0.33 in<sup>2</sup>/ft  
 Inside face        As4 = 0.20 in<sup>2</sup>/ft

**Top Slab:**

Outside face      As7 = 0.22 in<sup>2</sup>/ft  
 Inside face        As2 = 0.64 in<sup>2</sup>/ft

**Bottom Slab:**

Outside face      As8 = 0.24 in<sup>2</sup>/ft  
 Inside face        As3 = 0.70 in<sup>2</sup>/ft

**N. Check Shear**

**[5.14.5.3]**

**[12.5.5]**

**Sidewall**

The critical section for shear is taken at  $d_v$  from the tip of the haunch. The maximum design shear at this location is:

$$V_u = 4.48 \text{ kips with associated } M_u = 80.18 \text{ k-in}$$

**[5.8.3.3]**

The nominal shear resistance without the presence of shear reinforcement is given as:

$$V_r = \phi \cdot V_n$$

where:

$$V_n = \text{Lesser of } 0.25 \cdot f'_c \cdot b_v \cdot d_v \text{ or } V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v$$

The parameter,  $b_v$ , is the assumed member width and  $d_v$  is the effective shear depth.  $d_v$  is calculated as:

**[5.8.2.9]**

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.33 \cdot 65}{0.85 \cdot 5 \cdot 12} = 0.42 \text{ in}$$

$$d_v = \max(0.72h, 0.9d, d - a/2) = \max(0.72 \cdot 8, 0.9 \cdot 5.5, 5.5 - 0.42/2) \\ = \max(5.76, 4.95, 5.29) = 5.76 \text{ in} \quad \text{Use } d_v = 5.76 \text{ in}$$

MnDOT takes the shear resistance for box culverts to be the greater of that computed using LRFD Article 5.8.3.4.1 and 5.8.3.4.2. Using the "General Procedure", the crack spacing parameter,  $s_{xe}$ , is taken as:

**[5.8.3.4.2]**

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} = 5.76 \cdot \frac{1.38}{0.75 + 0.63} = 5.76 \text{ in and } 12 \text{ in} \leq s_{xe} \leq 80 \text{ in}$$

where:

$$s_x = d_v = 5.76 \text{ in}$$

$$a_g = \text{maximum aggregate size (in)} = 0.75 \text{ in}$$

Use  $s_{xe} = 12 \text{ in}$

$$\epsilon_s = \frac{\left( \frac{|M_u|}{d_v} + 0.5N_u + |V_u| \right)}{E_s \cdot A_s} = \frac{\left( \frac{|80.18|}{5.76} + 0.5 \cdot 0 + |4.48| \right)}{29000 \cdot 0.33} = 0.0019$$

where the magnitude of the moment,  $M_u$ , is not to be less than:

$$M_u \geq V_u \cdot d_v = 4.48 \cdot 5.76 = 25.8 \text{ k-in}$$

Because there is no shear reinforcement the value of  $\beta$  is taken as:

$$\beta = \frac{4.8}{1+750\varepsilon_s} \cdot \frac{51}{39 + s_{xe}} = \frac{4.8}{1+750 \cdot 0.0019} \cdot \frac{51}{39+12} = 1.98$$

AASHTO LRFD 5.8.3.4.1 allows a value of 2.0 to be used since the depth of the member is less than 16 in. and it is not subjected to axial tension. Therefore, use  $\beta=2.00$ .

### [5.8.3.3]

The factored shear resistance is then:

$$\begin{aligned}\phi V_c &= \phi \cdot 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v = 0.9 \cdot 0.0316 \cdot 2.0 \cdot \sqrt{5} \cdot 12 \cdot 5.76 \\ &= 8.79 \text{ kips}\end{aligned}$$

$$\begin{aligned}\phi V_n &= 8.79 \text{ kips} \leq \phi \cdot 0.25 \cdot f'_c \cdot b_v \cdot d_v \\ &= 0.9 \cdot 0.25 \cdot 5 \cdot 12 \cdot 5.76 = 77.8 \text{ kips} \quad \text{Use } 8.79 \text{ kips}\end{aligned}$$

$$\phi V_n = 8.79 \text{ kips} > V_u = 4.48 \text{ kips} \quad \text{OK}$$

### **Top Slab**

The maximum design shear at a distance  $d_v$  from the tip of the haunch is:

$$V_u = 6.61 \text{ kips with associated } M_u = 6.34 \text{ k-in}$$

### [5.14.5.3]

The shear resistance is:

$$\phi V_c = \phi \cdot \left[ 0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_e} \cdot \frac{V_u \cdot d_e}{M_u} \right] \cdot b \cdot d_e$$

where the quantity

$$\frac{V_u \cdot d_e}{M_u} \leq 1.0 \quad \frac{6.61 \cdot 6.5}{6.34} = 6.78 > 1.0 \quad \text{Use } 1.0$$

then

$$\phi V_c = 0.9 \cdot \left[ 0.0676 \cdot \sqrt{5} + 4.6 \cdot \frac{0.22}{12 \cdot 6.5} \cdot 1 \right] \cdot 12 \cdot 6.5 = 11.52 \text{ kips}$$

The shear capacity for the top slab cast monolithically with the sidewalls is not to be taken less than:

$$\begin{aligned}\phi V_c &= \phi \cdot 0.0948 \cdot \sqrt{f'_c} \cdot b \cdot d_e = 0.9 \cdot (0.0948 \cdot \sqrt{5} \cdot 12 \cdot 6.5) \\ &= 14.88 \text{ kips} > 11.52 \text{ kips}\end{aligned}$$

$$\phi V_c = 14.88 \text{ kips} > V_u = 6.61 \text{ kips} \quad \text{OK}$$

**Bottom Slab**

The maximum design shear at a distance  $d_v$  from the tip of the haunch is:

$$V_u = 7.36 \text{ kips with associated } M_u = 33.28 \text{ k-in}$$

**[5.14.5.3]**

The shear capacity is:

$$\phi V_c = \phi \cdot \left[ 0.0676 \cdot \sqrt{f'_c} + 4.6 \cdot \frac{A_s}{b \cdot d_e} \cdot \frac{V_u \cdot d_e}{M_u} \right] \cdot b \cdot d_e$$

where the quantity

$$\frac{V_u \cdot d_e}{M_u} \leq 1.0 \quad \frac{7.36 \cdot 7.5}{33.28} = 1.66 \geq 1.0 \quad \text{Use } 1.0$$

then

$$\phi V_c = 0.9 \cdot \left( 0.0676 \cdot \sqrt{5} + 4.6 \cdot \frac{0.24}{12 \cdot 7.5} \cdot 1.0 \right) \cdot 12 \cdot 7.5 = 13.24 \text{ kips}$$

The shear capacity for the bottom slab cast monolithically with the sidewalls is not to be taken less than:

$$\begin{aligned} \phi V_c &= \phi \cdot 0.0948 \cdot \sqrt{f'_c} \cdot b \cdot d_e = 0.9 \cdot (0.0948 \cdot \sqrt{5} \cdot 12 \cdot 7.5) \\ &= 17.17 \text{ kips} > 13.24 \text{ kips} \end{aligned}$$

$$\phi V_c = 17.17 \text{ kips} > V_u = 7.36 \text{ kips} \quad \text{OK}$$

**O. Check Thrust  
[5.7.4]**

The axial capacity of the culvert should be checked to ensure it satisfies the provisions of LRFD Article 5.7.4. The sidewall member will be checked since it has the largest thrust value and least amount of thickness. The design axial load is then:

$$P_u = 11.44 \text{ kips} \quad (\text{top, Strength Ia and Ib})$$

**[5.5.4.2.1]**

Without stirrups in the section, the resistance factor for compression is 0.70.

**[5.7.4.5]**

$$\phi P_n = \phi \cdot 0.10 \cdot f'_c \cdot A_g = 0.70 \cdot 0.10 \cdot 5 \cdot 8 \cdot 12 = 33.6 \text{ kips} > 11.44 \text{ kips} \quad \text{OK}$$

**[5.5.4.2.1]**

The axial capacity is adequate. MnDOT does not allow the consideration of the benefit from the applied axial force in computation of bending resistance of the sidewalls.

**P. Detailing/  
Reinforcing  
Lengths  
[5.11.2]  
[5.11.6]**

The concrete cover must be between 1½ inches minimum and 2 inches maximum. Also, the As1 reinforcing needs to be extended in the top and bottom slabs until the As7 or As8 reinforcing is adequate to resist the negative moment. In addition, the As7 and As8 reinforcement needs to be properly lapped to the As1 reinforcement to ensure reinforcement continuity. In this example As1 is not needed for shear resistance, so it does not need to be lapped past  $d_v$  from the tip of the haunch. For conservatism and simplicity of the design and construction, calculate development lengths and lap lengths on the bottom slab and then apply the longer computed length to both the top and bottom slabs. See Figure 12.2.3.7 for more detail. A summary of these calculations follows.

For As1, the reinforcing on the outside of the sidewalls, the area of steel required is 0.33 in<sup>2</sup>/ft. The development length, assuming the maximum, worst case wire spacing of 4 inches, is given as:

**[5.11.2.5.2-1]**

$$l_d = 8.50 \cdot \frac{A_w \cdot f_y}{s_w \cdot \sqrt{f'_c}} = 8.50 \cdot \frac{0.33 \cdot 65}{4 \cdot \sqrt{5}} = 6.79 \text{ in}$$

Since the minimum development length for smooth wire fabric is the embedment of two cross wires with the closer cross wire not less than 2 inches from the critical section, the minimum development length assuming 4 inch spacing is:

$$l_{d\min} = 4 + 4 + 2 = 10 \text{ in} > 6.79 \text{ in} \quad \text{Use 10 in}$$

**[5.11.6.2]**

For As8, the area of steel required is 0.24 in<sup>2</sup>/ft. The required lap length is given as the greater of  $1.5 \cdot l_d$  or 6 inches.

Then the minimum As8 lap length is

$$1.5 \cdot l_d = 1.5 \cdot 10 = 15 \text{ in} > 6 \text{ in} \quad \text{Use 15 in}$$

From the structural analysis software results, the distance to the point where the negative moment can be resisted by As8 is 0 inches. The lap length of 15 inches is used, since it is greater than the development length of As1 (10 in). The calculated M length is given as:

$$M = 0 + 15 = 15 \text{ in}$$

However, the minimum M length for the bottom slab based on MnDOT criteria is below. Note that 6 inches is added for consistency with past practice.

$$M_{\min} = T_s + \text{haunch} + \max(d_e, d_v) + 6" = 8 + 12 + 7.5 + 6 = 33.5 \text{ in. say } 2'-10"$$

The length of the As1 reinforcement is:

$$120 + 9 + 10 - 1.5 - 1.5 + 34 + 34 - 1.5 - 1.5 = 201 \text{ in. or } 16'-9"$$

The length of the As7 and As8 bars are then:

$$120 + 8 + 8 - 34 - 34 + 15 + 15 = 98 \text{ in. or } 8'-2"$$

The lengths of the As2, As3 and As4 bars are the span or rise plus 6 inches to ensure the bar is properly embedded into the member.

A summary of the reinforcing lengths is below.

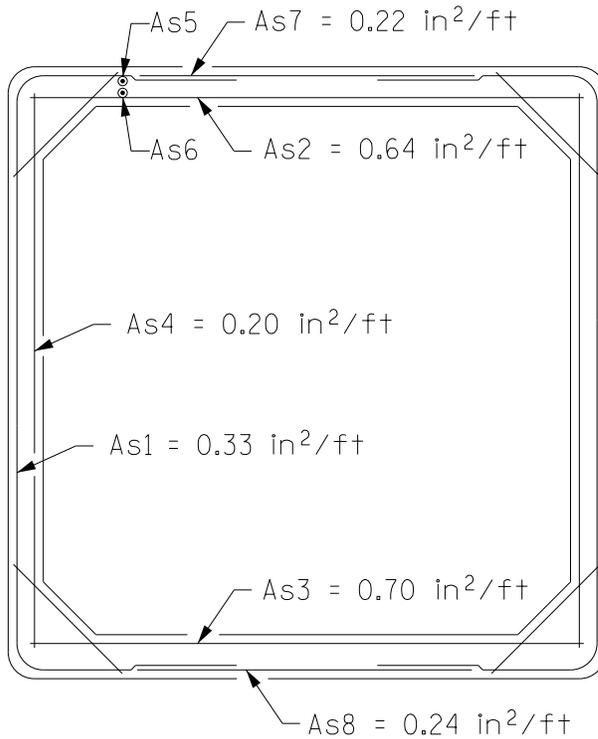
**Table 12.5.8**  
**Reinforcement Length Summary**

Reinforcement	Length
"M" Dimension	2'-10"
As1	16'-9"
As2	10'-6"
As3	10'-6"
As4	10'-6"
As7	8'-2"
As8	8'-2"

**Q. Summary**

Figure 12.5.6 illustrates the required reinforcing for the inside face and outside face of the sidewalls, top slab, and bottom slab. Longitudinal steel area is 0.06 in<sup>2</sup>/ft.

Note that if reinforcing bars are used rather than welded wire fabric, the required reinforcement must be increased by a factor of 65/60 = 1.08 to account for the difference in yield strength. Also, crack control must be rechecked.



**Figure 12.5.7**  
**Box Culvert Reinforcement**

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**12.6 16'x12'  
Precast Concrete  
Box Culvert Live  
Load Distribution  
Example**

This example illustrates the computation of live load to a precast box culvert with a 16 foot span under 1 foot of fill. The culvert has a top slab thickness of 12 inches, bottom slab thickness of 11 inches and sidewall thicknesses of 8 inches. For an example of all other loading calculations, analysis, design, or detailing, see Article 12.5 of this manual.

**A. Live load  
[3.6.2.2]  
[3.6.2.2-1]**

**Dynamic Load Allowance**

$$IM = 33 \cdot [1.0 - 0.125 \cdot D_E] = 33 \cdot [1.0 - 0.125 \cdot 1.0] = 28.9\%$$

**[3.6.1.2.5]  
[3.6.1.2.6]  
[4.6.2.10]**

**Live Load Distribution**

Since the depth of fill is less than 2 feet, live loads are distributed using an equivalent strip width.

A single loaded lane with the single lane multiple presence factor is analyzed. Assuming traffic travels primarily parallel to the span, the axle loads are distributed to the top slab accordingly.

Perpendicular to the span:

$$E = 96 + 1.44 \cdot S = 96 + 1.44 \cdot 16 = 119.04 \text{ in}$$

Parallel to the span:

$$E_{\text{span}} = L_T + LLDF \cdot H = 10 + 1.15 \cdot 12 = 23.8 \text{ in}$$

where:

- E = Equivalent distribution width perpendicular to span (in)
- E<sub>span</sub> = Equivalent distribution length parallel to span (in)
- L<sub>T</sub> = Length of tire contact area parallel to span (in)
- LLDF = 1.15, factor for distribution of live load through depth of fill
- H = Depth of fill from top of culvert to top of pavement (in)
- S = Clear span (ft)

AASHTO Article 4.6.2.10.4 states that the load distribution width shall not exceed the length between the adjacent joints without a means of shear transfer across the joint. Since this culvert has less than 2 feet of fill, MnDOT requires a distribution slab. A distribution slab is considered to be a means of shear transfer across the box culvert joints, so in this example the load distribution width is not limited to the section length and the full width of 9.92' can be used.

**[3.6.1.1]**

A single HL-93 truck axle configuration produces a live load intensity of:

$$w_{LL+IM} = \frac{2 \cdot P_w \cdot MPF \cdot (1+IM)}{\text{Influence Area}} = \frac{2 \cdot 16 \cdot 1.2 \cdot (1+0.289)}{9.92 \cdot 1.98} = 2.52 \text{ klf}$$

A single tandem vehicle produces a live load intensity of:

$$w_{LL+IM} = \frac{4 \cdot P_w \cdot MPF \cdot (1+IM)}{\text{Influence Area}} = \frac{4 \cdot 12.5 \cdot 1.2 \cdot (1+0.289)}{2 \cdot 9.92 \cdot 1.98} = 1.97 \text{ klf}$$

where:

MPF = 1.2 Multiple Presence Factor for one lane

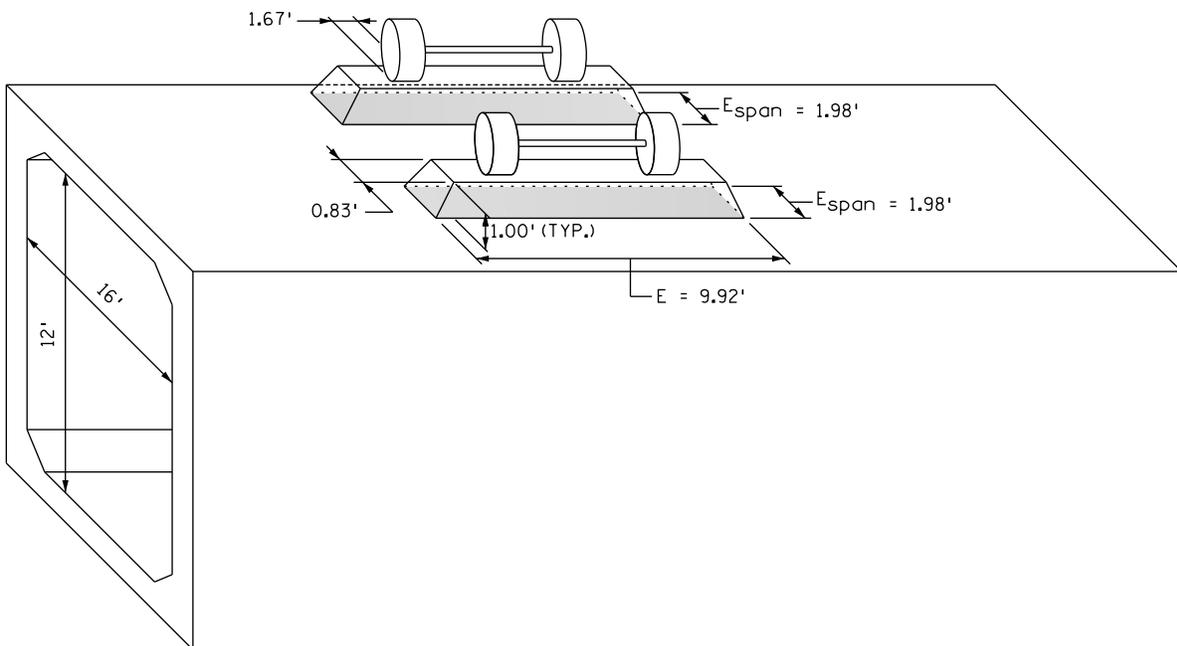
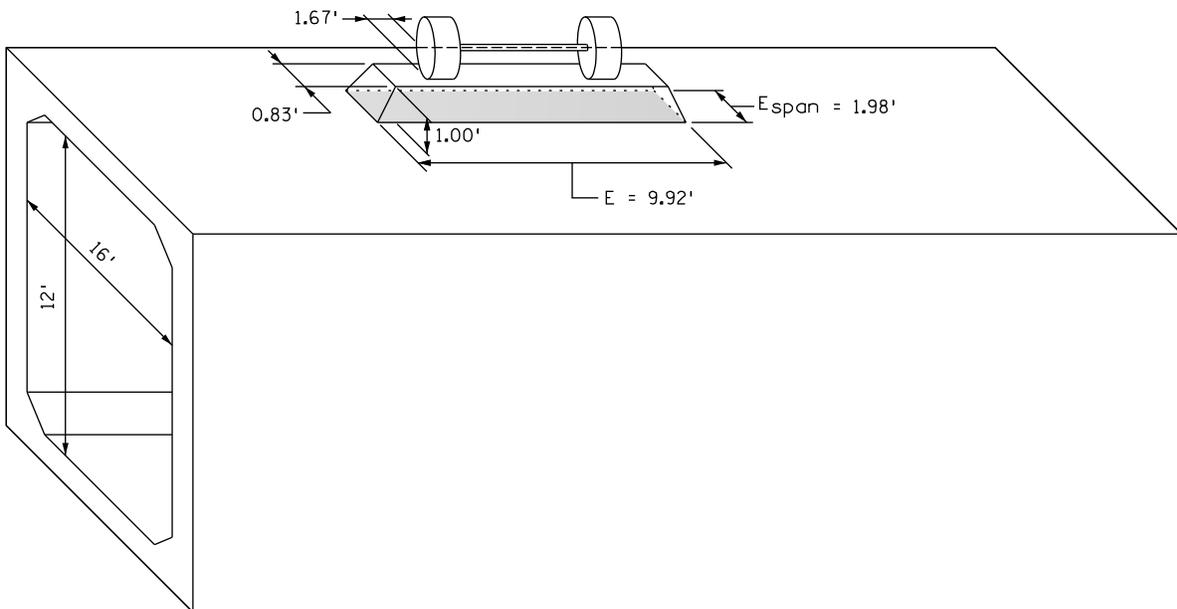
$P_w$  = Wheel load for design vehicle (kips)

**[3.6.1.1.2]****[3.6.1.2.2]****[3.6.1.2.3]****[3.6.1.2.4]**

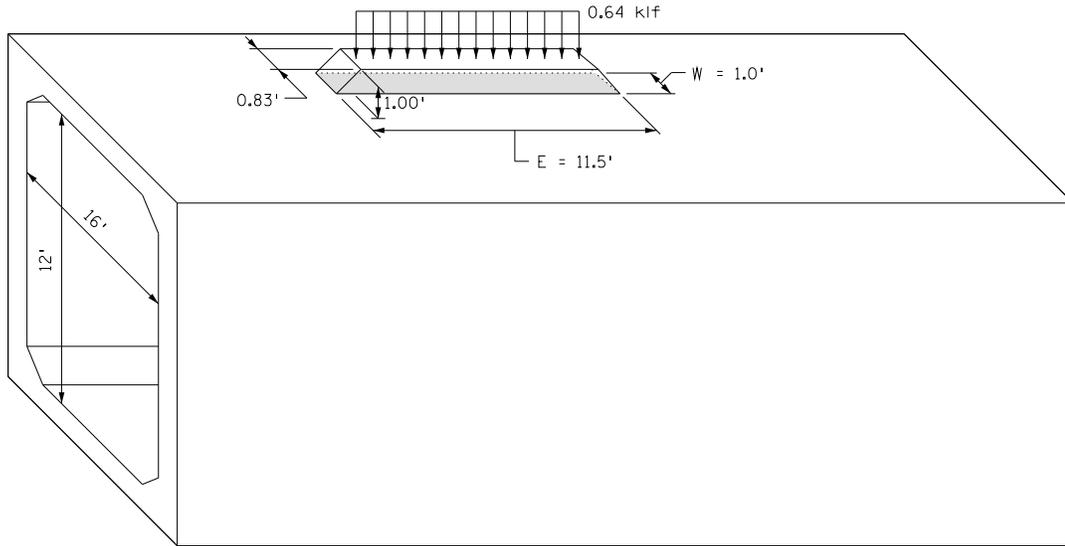
The design lane load is a 0.64 klf load uniformly distributed in the longitudinal direction and assumed to be distributed uniformly over ten feet in the transverse direction. The lane load is not subjected to a dynamic load allowance.

$$w_{\text{Lane}} = \frac{0.64}{10 \cdot 1.15} = 0.056 \text{ klf}$$

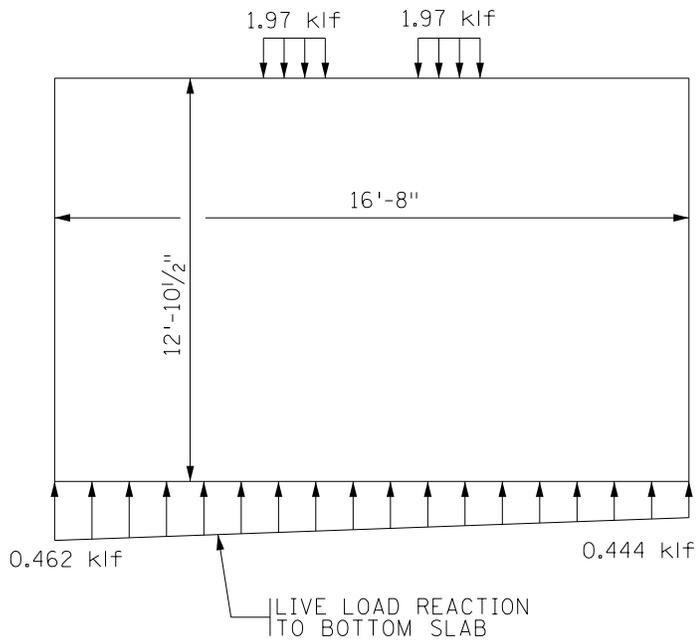
The following figures illustrate the different live loads and how they are applied to the box culvert with less than 2 feet of fill.



**Figure 12.6.1**  
**Live Load Distribution, Single HL-93 Truck and Tandem Axle Configurations**



**Figure 12.6.2**  
**Live Load Distribution, HL-93 Lane Load**



**Figure 12.6.3**  
**Live Load Distribution, Single HL-93 Tandem Applied to Top and Bottom Slabs**

### **13. RAILINGS**

Section 13 of the LRFD Specifications addresses the design of railings. "Railings" is used as a generic term in the specifications. Railings include traffic safety barriers as well as median barriers, bicycle, and pedestrian railings.

The design requirements for railings utilized on Mn/DOT bridges have undergone changes in recent years as the Federal Highway Administration (FHWA) established crash-testing requirements and the AASHTO Specifications were revised accordingly. Additionally, the desire for more attractive railings has influenced the style of railings on projects where aesthetics is a major consideration. Accidents involving objects thrown from overpasses onto traffic below has led to the adoption of protective screening requirements. The rapid increase in bicycle trails and traffic has increased attention on bicycle railings. This section of the LRFD Bridge Design Manual details our policies regarding the design of bridge railings for Mn/DOT projects.

#### **13.1 Materials**

Reinforced concrete, steel, and timber are all used for railings. The majority of traffic railings are reinforced concrete. Bridges with timber decks on low volume secondary roads may have timber railings. Pedestrian and bicycle railings are typically galvanized steel that has been painted for aesthetics.

#### **13.2 Design Requirements**

The design of newly constructed bridge railings must conform to the requirements of Section 13 of the *AASHTO LRFD Bridge Design Specifications*. This specification gives geometric and strength requirements and also describes crash test levels. FHWA requires all bridges carrying traffic on the National Highway System (NHS) to be crash tested in accordance with *NCHRP Report 350 Recommended Procedures for the Safety Performance Evaluation of Highway Features*. There are six levels of service and testing depending on vehicle size and speed. A list of crash tested railings is found on the following FHWA Web sites:

- <http://www.fhwa.dot.gov/bridge/bridgerail/>
- [http://safety.fhwa.dot.gov/roadway\\_dept/road\\_hardware/bridgerailings.htm](http://safety.fhwa.dot.gov/roadway_dept/road_hardware/bridgerailings.htm)
- [http://safety.fhwa.dot.gov/roadway\\_dept/road\\_hardware/longbarriers.htm](http://safety.fhwa.dot.gov/roadway_dept/road_hardware/longbarriers.htm)

Crash testing requirements may be waived if the railing in question is similar in geometrics to an approved crash tested rail and an analytical evaluation shows the railing to be crash worthy. This allows minor changes to crash tested railings without having to go through the time and expense of crash testing. For bridges on the NHS any such evaluation must be approved by the FHWA.

Crash testing has shown that during impact vehicles slide along the top of the railing and parts of the vehicle, especially the boxes on trucks, extend beyond the face of the railing a considerable distance. The envelope of the vehicle encroachment beyond the face of railing is known as the zone of intrusion. Attachments to bridge railings, such as architectural metal railings or objects just behind the railing (such as light poles), must address safety concerns presented by this encroachment, which include:

- 1) Snagging - which can cause the attachment or the vehicle hood to penetrate the occupant compartment.
- 2) Spearing – objects, such as a horizontal railing member, penetrating windshields and injuring occupants.
- 3) Debris falling onto traffic below.

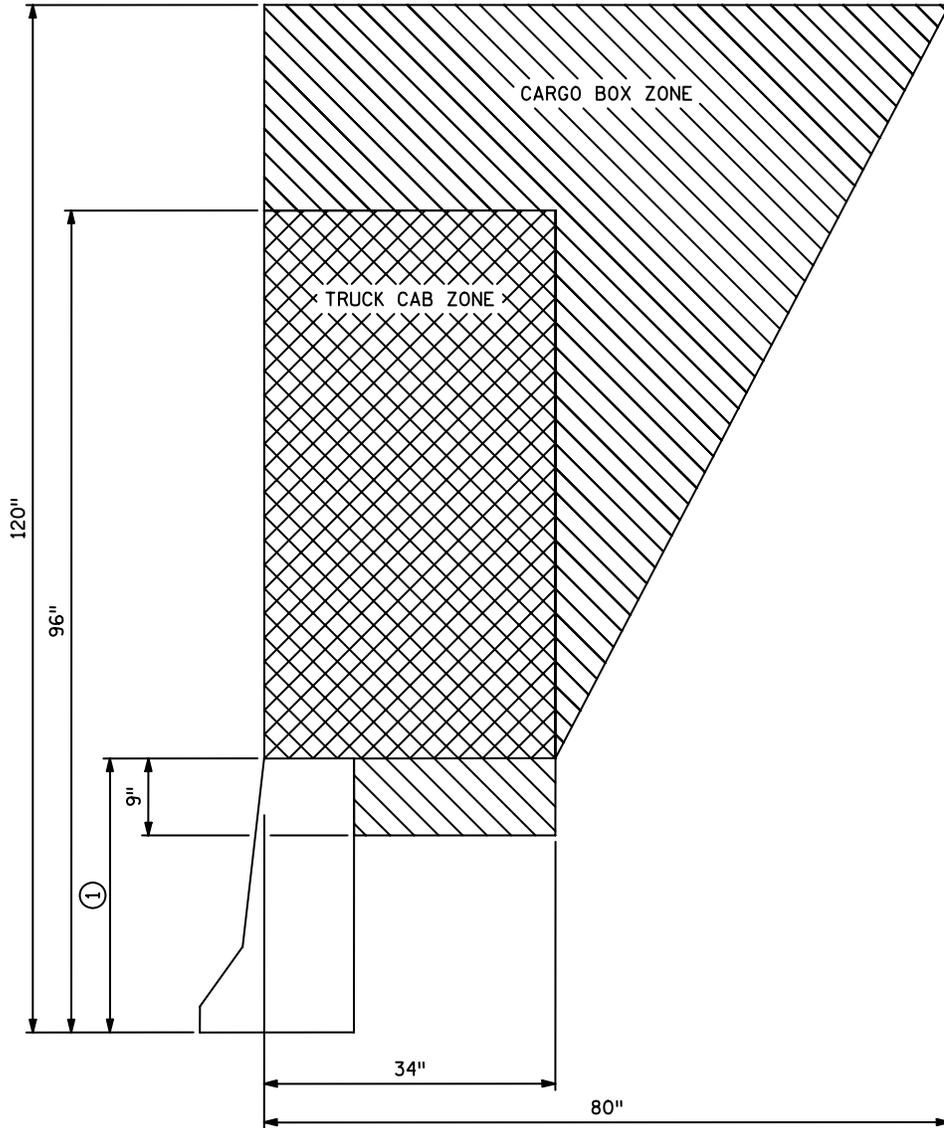
A Midwest Roadside Safety Facility report, titled *Guidelines for Attachment to Bridge Rails and Median Barriers*, February 26, 2003, identifies zones of intrusion for several types of railings. Figure 13.2.1 shows the zone of intrusion for a Test Level 4 barrier.

Generally attachments within the zone of intrusion shall be designed to break away before severely damaging the vehicle, contain any debris from damaging traffic below, and have no members (such as rail ends) that might spear the occupant compartment of the vehicle. Ends of rails shall be sloped at 45 degrees or less to top of barrier to reduce the chance of spearing. Posts shall be set back from the face of railing to minimize snagging. (See Sections 13.2.1 and 13.2.3 for setback requirements.)

Railing designs shall include consideration of safety, cost, aesthetics and maintenance. Safety shapes (Types J and F) were developed to minimize damage to vehicles, as well as to contain and redirect vehicles back onto the roadway, and have low initial and maintenance costs. Use of designs that allow for easy replacement of damaged sections and use of standard railings can minimize maintenance costs since replacement components can be stockpiled.

Three general classes of bridge railings are Traffic Railings, Pedestrian or Bicycle Railings, and Combination Railings. Bridge cross sections showing

these three classes are shown in Figure 13.2.2. Railing classes are further defined in the following sections. Also, refer to Table 13.2.1 for guidance on standard rail applications.



① REVIEWED TL-4 BARRIER HEIGHTS FELL IN A RANGE OF 29" TO 42"

<sup>1</sup> **Figure 13.2.1**  
**Intrusion Zones for TL-4 Barriers**

<sup>1</sup> Reproduced from Keller, Sicking, Faller, Polivka & Rhode, *Guidelines for Attachments to Bridge Rails and Median Barriers*, (Midwest Roadside Safety Facility, February 26, 2003), page 24.

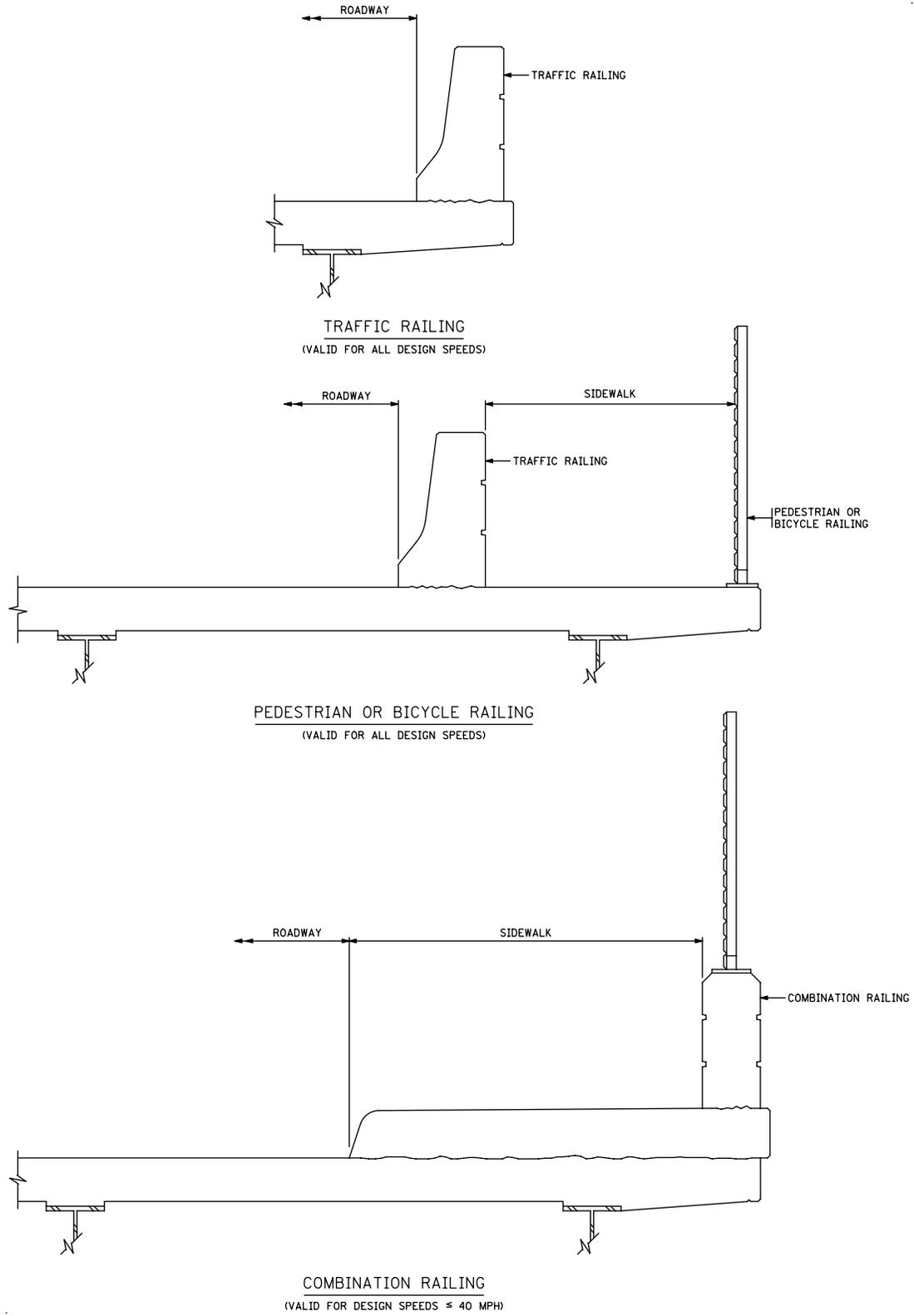


Figure 13.2.2

TABLE 13.2.1: Standard Rail Applications

Rail Type	Description	Test Level	Speed Limit	Application	Comment
Traffic	Concrete Barrier (Type F, TL-4) 5-397.114: Separate End Post w/o W.C. 5-397.115: Integral End Post w/o W.C. 5-397.116: Separate End Post w/ W.C. 5-397.117: Integral End Post w/ W.C.	TL-4	All	Traffic Only	2'-8" tall
	Concrete Barrier (Type F, TL-5) 5-397.122: Integral End Post w/ W.C. 5-397.124: Integral End Post w/o W.C.	TL-5	> 40 mph	High Protection Area where $D_c > 5^\circ$ and Speed > 40 mph.	3'-6" tall (Gives added protection to motorists on high speed, high curvature roadways. Modify standard to remove sidewalk.)
	Concrete Barrier (Type F, TL-5) w/ Sidewalk 5-397.125: Integral End Post w/ W.C. 5-397.126: Integral End Post w/o W.C.	TL-5	All	Between sidewalk and roadway where the shoulder is < 6'.	3'-6" tall (The additional height is to protect a bicycle rider from falling over the railing into traffic.)
	Concrete Barrier (Type F, TL-5) 5-397.128: Integral End Post w/ W.C. 5-397.129: Integral End Post w/o W.C.	TL-5	All	Bridges with designated bike path or where glare screen is required.	4'-8" tall
	Solid Median Barrier (Type F, TL-4) 5-397.130: w/ W.C.	TL-4	All	Traffic Only	2'-8" tall
	Split Median Barrier (Type F, TL-4) 5-397.131: w/ W.C.	TL-4	All	Bridges with a longitudinal joint between roadways. (Usually the bridge is very wide or is to be constructed in stages.)	2'-8" tall (For stage construction, each half of barrier meets TL-4 standard.)
	Solid Median Barrier and Glare Screen (Type F, TL-4) 5-397.132: w/ W.C.	TL-4	All	Traffic Only	4'-8" tall
	Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.135: w/o W.C. 5-397.136: w/ W.C.	TL-4	All	Traffic Only	4'-8" tall
	Offset Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.137: w/ W.C.	TL-4	All	Use where roadways are at different elevations. (Usually on superelevated bridges.)	4'-8" tall (Separation allows both sides to be slipformed.)

TABLE 13.2.1: Standard Rail Applications (cont.)

Rail Type	Description	Test Level	Speed Limit	Application	Comment
Traffic	Concrete Barrier (Type P-2, TL-4) and Structural Tube Railing (Type T-1) 5-397.157: w/ Integral End Post	TL-4	All	Traffic Only, where an aesthetic railing is desired.	1'-3" metal railing on 1'-9" parapet (Designer must modify detail for separate end post or no W.C.)
	Concrete Barrier (Type P-4, TL-4) 5-397.173: Integral End Post w/ W.C.	TL-4	All	Traffic Only	2'-8" tall
Combination (Traffic and Ped./Bicycle)	Concrete Barrier (Type P-1, TL-2) and Wire Fence (Design W-1) 5-397.119: Integral End Post 5-397.120: Separate End Post	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	2'-4" parapet and 6' metal rail with chain link fabric.
	Concrete Barrier (Type P-1, TL-2) and Metal Railing for Bikeway (Type M-1) 5-397.154: Integral End Post	TL-2	≤ 40 mph	Outside edge of walk on highway bridges with sidewalks where bicycle traffic on the walk is expected and protective screening is not required.	2'-4" parapet with 2'-2" metal rail (Modify for separate end post.)
	Structural Tube Railing (Design T-2) 5-397.158	TL-4	All	Attachment to Type F rail for use where significant bicycle traffic will be using roadway shoulder.	Top of metal railing 1'-10½" above top of 2'-8" Type F rail (Total height of 4'-6"± meets bicycle standard.)
	Concrete Barrier (Type P-1, TL-2) and Structural Tube Railing with Fence (Design T-3) 5-397.212	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	2'-4" parapet and 5'-8 ½" metal rail with chain link fabric
Ped./Bicycle	5' Wire Fence (Design W-1) for Pedestrian Bridges 5-397.202	Ped. & Bike	Yes	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	5' tall chain link fence
	8' Wire Fence for Pedestrian Walks 5-397.205	Ped. & Bike	Yes	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	8' tall chain link fence

NOTES:

- Crash testing levels refer to NCHRP Report 350. The structural tube traffic rail (Bridge Details Manual Part II, Fig. 5-397.157) and bicycle rail attachment to Type F rail (Bridge Details Manual Part II, Fig. 5-397.158) were developed by Minnesota and crash tested through the pooled fund program. Combination railings with the 2'-4" parapet have been judged to meet crash Test Level 2 (TL-2) by comparison to other crash tested vertical face railings. Railing heights are measured to the finished surface (top of wearing course).
- Information on current costs of these railings may be obtained from the Bridge Estimating Unit.
- Combination railings may also be used as bicycle/pedestrian railings. The 2'-4" parapet height permits a wider spacing of spindles (6" openings rather than the 4" openings required up to 27" above the finished surface).

TABLE 13.2.2: Non-Standard Rail Applications

Rail Type	Description	Test Level	Speed Limit	Application	Comment
Combination (Traffic and Ped./Bicycle)	Cloquet Railing Bridge No. 09008 and 09009	TL-2	≤ 40 mph	Outside edge of walk on highway bridges with sidewalks where bicycle traffic on the walk is expected and protective screening is not required.	2'-2 3/4" metal rail on 2'-4" parapet (Sheet is metric.)
	Concrete Barrier (Type P-3, TL-2) and Ornamental Metal Railing (Type M-2)	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	3'-9" metal rail on 2'-4" parapet (Developed by City of Minneapolis for use on bridges in their city.)
	St. Peter Railing Bridge No. 27R05	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	4'-6" metal rail on 2'-4" parapet (Bridge No. 23022 has a 2'-2" height of metal rail for use where protective screening is not needed.)
	TH 100 Corridor Standard Bridge No. 27285	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	3'-9" metal rail on 2'-4" parapet
	TH 212 Corridor Standard Bridge No. 27148	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	5'-8" to 9'-2" metal rail on 2'-4" parapet
	TH 610 Corridor Standard Ornamental Metal Railing Type DWG Bridge No. 27222	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	5'-5 1/2" metal rail on 2'-4" parapet (Sheet is metric.)
	Victoria Street Railing Bridge No. 62823	TL-2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	5'-8" metal rail on 2'-4" parapet with chain link fabric

TABLE 13.2.2: Non-Standard Rail Applications (cont.)

Rail Type	Description	Test Level	Speed Limit	Application	Comment
Pedestrian/Bicycle	Gooseberry Falls Suspended Walkway Rail Bridge No. 38010	Ped.	N/A	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	3'-6" tall (Sheet is metric.)
	Lexington Rail Bridge No. 62823	Ped. & Bike	N/A	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	4'-6" tall (Sheet is metric.)
	St. Peter Rail Bridge No. 40002	Ped. & Bike	N/A	Pedestrian bridges or sidewalks separated from roadways by a traffic barrier.	4'-6" tall (Sheet is metric.)

NOTES:

- Crash testing levels refer to NCHRP Report 350. Combination railings with the 2'-4" parapet have been judged to meet crash Test Level 2 (TL-2) by comparison to other crash tested vertical face railings.
- Railing heights are measured to the finished surface (top of wearing course).
- Information on current costs of these railings may be obtained from the Bridge Estimating Unit.
- Combination railings may also be used as pedestrian/ bicycle railings. The 2'-4" parapet height permits a wider spacing of spindles (6" openings rather than the 4", which is required in the lower 27").

### 13.2.1 Traffic Railing

Traffic railings are designed to contain and safely redirect vehicles. Requirements based on speed are as follows.

1) High Speed Roadways with a Design Speed > 40 mph

Mn/DOT requires crash testing to Test Level 4 as the minimum standard for these roadways. Test Level 4 is run with a small car and a pickup truck at 60 mph and a single unit van truck impacting at 50 mph. This railing will normally be the 32" high Type F barrier (Bridge Details Manual Part II, Figure 5-397.114-117). Where aesthetic needs warrant, the tubular traffic railing (Bridge Details Manual Part II, Figure 5-397.157) is an acceptable alternative that provides an increased viewing opportunity to drivers crossing the bridge. It consists of a structural tube and posts mounted to the top of a 1'-9" high concrete base. Note, however, that the tubular traffic railing has higher initial and maintenance costs than the Type F barrier. Consult the Preliminary Bridge Unit for additional acceptable railings.

Mn/DOT has developed a bicycle railing attachment to the Type F barrier for use where the bridge shoulders carry a bicycle route as defined in the Mn/DOT State Bicycle Transportation System Plan or another recognized authority. This attachment (Bridge Details Manual Part II, Figure 5-397.158) adds height to the railing to protect bicycle riders and has been crash tested to Test Level 4. It has a cable system inside the rail tubes that will contain the rail pieces in the event of an accident. It also uses weakened posts designed to lessen the impact to vehicles in the event of a hit. This railing may be applied to other traffic barriers provided that the same or greater offset distance to the face of metal rail is provided and the post attachment has the same or greater strength. The cable system must be maintained even if there is no traffic below as the cables act to keep the entire rail system intact during a crash.

The zone of intrusion (see Section 13.2 for definition) shall be kept free of rail attachments or other features unless they have been crash tested or an analytical evaluation has shown them to be crash worthy. Exceptions to this policy include noise walls and safety features such as signs or lights. Note that light poles shall be located behind the back of the barrier. When noise walls are attached, consider using a higher Type F barrier to lessen the risk. The zone of intrusion for a TL-4 railing is shown in Figure 13.2.1.

A more stringent rail design may be considered on a case-by-case basis for bridges with high design speeds, high truck volume, and curvature or other site-specific safety considerations. Generally a Test Level 5 railing should be considered for these sites. Test Level 5

includes a small car and a pickup truck traveling at 60 mph plus a van-type tractor trailer impacting at 50 mph. As a guide, a 42" high Type F barrier that meets TL-5 requirements is recommended for bridges having a horizontal curvature of 5 degrees and sharper on a roadway where the design speed is 45 mph or higher. The Preliminary Bridge Plans Engineer will designate the rail design on the Preliminary Bridge Plan.

2) Low Speed Roadways with a Design Speed  $\leq$  40 mph

Mn/DOT requires crash testing to Test Level 2 as the minimum standard for these roadways. Test Level 2 is run with a small car and pickup truck both impacting at a speed of 45 mph.

Normally these railings will be the same as used for higher speeds, usually the Type F concrete barrier, but with the reduced level required for crash testing more options are available. Consult the Preliminary Bridge Unit for additional acceptable railings.

If the addition of an ornamental metal railing is desired on the top of the traffic railing, a 32" high vertical faced concrete barrier (see Bridge Details Manual Part II, Figure 5-397.173) shall be used rather than the Type F barrier. The vertical face will cause more damage to a vehicle for minor hits but reduces the tendency for the vehicle to climb the face or roll over and will keep the vehicle back from the metal rail. A small 2" wide by 6" high curb is provided at the base to minimize snowplow damage to the barrier. For design speeds of 35 mph and below a metal railing may be used on the top of the concrete barrier with no minimum offset required, as it is unlikely that vehicles will contact the metal portion.<sup>2</sup> With a design speed of 40 mph the front face of the metal railing shall be offset a minimum of 9" from the face of barrier at the top of concrete.<sup>3</sup>

It is strongly recommended that a smooth face be used on the highway side of concrete barriers. Aesthetic treatments on the highway face increase the risk of vehicle snagging. In addition, in this environment the aesthetics treatment will routinely experience vehicle hits, snowplow scrapes, and high exposure to salt. As a result, their performance will be greatly reduced, causing increased maintenance costs.

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<sup>2</sup> Reproduced from Keller, Sicking, Faller, Polivka & Rhode, *Guidelines for Attachments to Bridge Rails and Median Barriers*, (Report dated February 26, 2003), pages 3 and 27.

<sup>3</sup> Reproduced from Keller, Sicking, Faller, Polivka & Rhode, *Guidelines for Attachments to Bridge Rails and Median Barriers*, (Report dated February 26, 2003), page 15 and 16. 9" offset at 40 mph judged acceptable based on 12" offset at 45 mph.

**13.2.2 Pedestrian/  
Bicycle Railing**

Pedestrian or bicycle railings are generally located at the outside edge of a bridge sidewalk and are designed to safely contain pedestrians or bicyclists. AASHTO specifications require pedestrian railings to be at least 3'-6" in height and bicycle railings to be at least 4'-6" in height. The height is measured from the top of walkway to top of the highest horizontal rail component.

Openings between members of a pedestrian railing shall not allow a 4" sphere to pass through the lower 27" of the railing and a 6" sphere should not pass through any openings above 27". This is more restrictive than AASHTO and is intended to prevent small children from slipping through the railing. The International Building Code requires a 4" maximum opening.

**13.2.3  
Combination  
Railing**

Combination railings are dual purpose railings designed to contain both vehicles and pedestrians or bicycles. These railings are generally located at the outside edge of a bridge sidewalk. A raised sidewalk is used to clearly define the walkway area and keep roadway drainage off the walkway. The sidewalk curb offers some protection to pedestrians from errant vehicles entering the walkway. There is no other barrier between the roadway and the sidewalk. Combination railings are applicable for design speeds of 40 mph and under. Mn/DOT requires crash testing to Test Level 2 for these railings and the strength and geometrics requirements for bicycle or pedestrian railings also apply.

Combination railings will normally consist of a 2'-4" high concrete parapet with a fence or ornamental metal railing mounted on the top. The concrete parapet serves to contain traffic and has been judged to meet crash Test Level 2. The metal railing must comply with the strength and geometric requirements for bicycle or pedestrian railings. A non-crash tested metal railing may be used on the top of the concrete barrier, as it is unlikely that vehicles will make contact with the metal portion.

For typical applications, the highway face of a concrete parapet shall be relatively smooth for ease of construction (slipforming) and maintenance. Where aesthetic needs warrant it, beveled recesses up to 2" deep may be allowed for inset panels and beveled form liner textures. Concrete posts above the parapet are acceptable but they may not project in front of the parapet.

For design speeds greater than 40 mph, a traffic railing is required between the roadway and sidewalk or bikeway. Use a 32" high Type F barrier for the traffic railing when the shoulder is 6'-0" or greater in width. If the roadway shoulder is less than 6'-0", use a 42" Type F

barrier for added protection. Metal railings shall not be placed on top of a traffic railing between a sidewalk and a roadway. Although metal railings may somewhat increase protection for bicyclists, they are a risk hazard to vehicles.

**13.2.4 Strength of  
Standard Concrete  
Barriers**

Barrier resistance values have been determined for the standard Mn/DOT concrete barriers and are shown in Table 13.2.4.1. They are based on using both near and far face reinforcement as tension reinforcement. These values can be used when analyzing deck overhangs to determine reinforcement requirements. (See Section 9.2.4J for an overhang reinforcement design example.)

TABLE 13.2.4.1: Resistance Values for Standard Concrete Barriers

Description	End Panel		Interior Panel	
	L <sub>c</sub> (ft)	R <sub>w</sub> (kips)	L <sub>c</sub> (ft)	R <sub>w</sub> (kips)
Concrete Barrier (Type F, TL-4) 5-397.114: Separate End Post w/o W.C. 5-397.115: Integral End Post w/o W.C.	4.6	59.2	9.9	124.1
Concrete Barrier (Type F, TL-4) 5-397.116: Separate End Post w/ W.C. 5-397.117: Integral End Post w/ W.C.	4.6	57.2	10.2	122.9
Concrete Barrier (Type F, TL-5) 5-397.122: Integral End Post w/ W.C.	9.3	128.5	14.3	128.8
Concrete Barrier (Type F, TL-5) 5-397.124: Integral End Post w/o W.C.	9.2	133.6	14.0	131.4
Concrete Barrier (Type F, TL-5) w/ Sidewalk 5-397.125: Integral End Post w/ W.C.	9.3	128.5	14.3	128.8
Concrete Barrier (Type F, TL-5) w/Sidewalk 5-397.126: Integral End Post w/o W.C.	9.2	133.6	14.0	131.4
Concrete Barrier and Glare Screen (Type F, TL-5) 5-397.128: Integral End Post w/W.C.	9.3	128.5	14.3	128.8
Concrete Barrier and Glare Screen (Type F, TL-5) 5-397.129: Integral End Post w/o W.C.	9.2	133.6	14.0	131.4
Split Median Barrier (Type F, TL-4) 5-397.131: w/ W.C.	4.5	54.0	12.1	91.1
Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.135: w/o W.C.	4.1	55.8	9.0	106.6
Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.136: w/ W.C.	4.2	61.1	9.2	107.5

TABLE 13.2.4.1: Resistance Values for Standard Concrete Barriers

Description	End Panel		Interior Panel	
	L <sub>c</sub> (ft)	R <sub>w</sub> (kips)	L <sub>c</sub> (ft)	R <sub>w</sub> (kips)
Offset Split Median Barrier and Glare Screen (Type F, TL-4) 5-397.137: w/ W.C.	4.2	61.1	9.2	107.5
Concrete Barrier (Type P-2, TL-4) 5-397.157: w/ Integral End Post	4.6	87.7	9.0	196.7
Concrete Barrier (Type P-4, TL-4) 5-397.173: Integral End Post w/ W.C.	4.6	76.8	9.9	151.7
Concrete Barrier (Type P-1, TL-2) 5-397.119 5-397.120 5-397.154 5-397.212	4.9	50.4	9.2	103.7

### **13.2.5 Protective Screening**

The addition of protective screening to bridge railings is a further Mn/DOT policy requirement. The practice of adding protective screening is common nationwide in response to accidents and fatalities that have occurred due to pedestrians throwing objects from overpasses onto vehicles below.

Protective screening must be included in the design of new bridges that accommodate pedestrians when the bridge crosses a roadway or railroad, and also when railings are replaced on existing bridges as follows:

- On bridges where a sidewalk is included in the design, incorporate a protective screening system in the design of the railing adjacent to the sidewalk.
- On pedestrian bridges, place the protective screening on both sides of the bridge.

The protective screening system will be, preferably, a chain link fence system or a railing system. The height of the fence or railing shall be 8'-0" above the top of the sidewalk. For sites with special aesthetic treatments involving ornamental railings a minimum height of 6'-0" will be allowed. However, it should be recognized that the lower railing height provides a reduced level of protection. The protective screening system shall not allow objects 6" or greater in diameter to pass through the fence or railing.

### **13.2.6 Architectural/Ornamental Railings**

In response to local requests, special railing designs have been incorporated in some projects to address aesthetic concerns. These ornamental architectural bridge railings have been utilized in lieu of standard combination railings for placement on the outboard side of bridge sidewalks. The Bridge Office will consider railing designs in addition to our standard railings for such locations and corridors. It is recommended that special railings incorporate features from the standard railings (such as connection details) as significant effort has gone into the development of these details.

Mn/DOT participation in the cost of aesthetic railings is governed by the Mn/DOT Policy Manual of June 2001. Refer to these documents for more information:

- Guidelines: *Mn/DOT Policy and Procedures for Cooperative Construction Projects with Local Units of Government*
- Position Statement: *Mn/DOT Policy and Procedures for Cooperative Construction Projects with Local Units of Government*

Railings are included with other aesthetic costs of the bridge. Mn/DOT participation is limited to 5%, 7% or 15% of the cost of a basic bridge, depending on the aesthetic level of the bridge.

Cost participation of architectural/ornamental railings on local bridges is generally funded up to the prorated cost of standard railing or chain link fence. Consult the State-Aid for Local Transportation Office for conditions on bridge funding eligibility.

### ***13.3 Design Examples***

Two design examples follow. The first illustrates the design procedures associated with a conventional Type F barrier. The second design example illustrates the steps undertaken for the design of adhesive anchors to support a metal railing.

**13.3.1 Type F  
Barrier Design  
Example**

This example illustrates a design check of the vertical reinforcing steel that ties a standard Mn/DOT Type F barrier to a concrete deck. The geometry of the barrier and the reinforcing bar sizes and types are illustrated in Bridge Details Part II Fig. 5-397.117. The configuration of the horizontal reinforcing bars in the railing is assumed fixed. The spacing of the vertical reinforcing steel is checked to ensure adequate capacity is provided. The design check uses the method described in LRFD Article A13.3.1.

**A. Design Forces  
and Dimensions**

**[13.7.3.2]**

Mn/DOT's Type F barrier satisfies the geometric height constraint of a TL-4 barrier and has satisfactorily passed crash testing to such a level. The design forces and dimensional limits for a TL-4 barrier presented in LRFD Table A13.2-1 are repeated below.

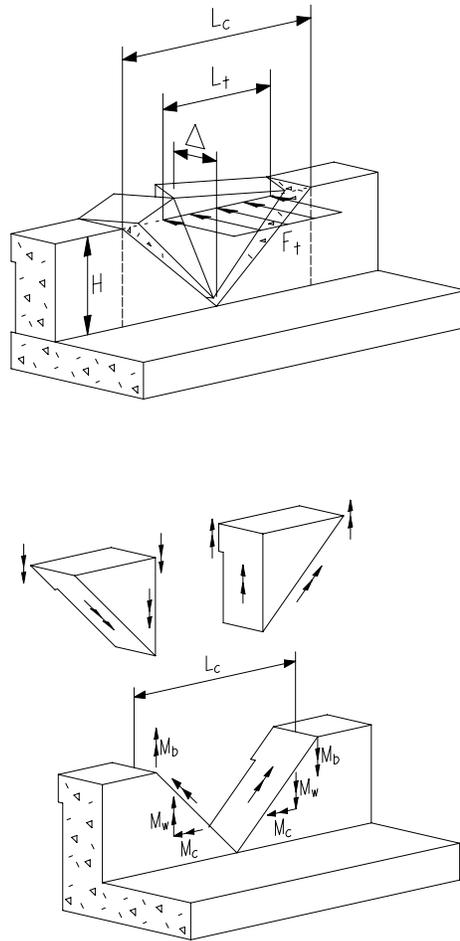
Design Forces and Designations	TL-4 Barrier
$F_t$ Transverse (kip)	54
$F_L$ Longitudinal (kip)	18
$F_V$ Vertical/Down (kip)	18
$L_t$ and $L_L$ (ft)	3.5
$L_V$ (ft)	18
$H_e$ Minimum Height of Horizontal Loads (in)	32
H Minimum Height of Rail (in)	32

The design is based on yield line analysis methods and has three variables:

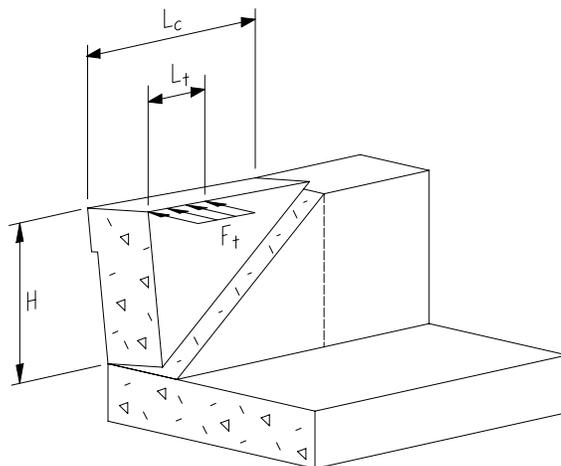
- $M_b$  – the flexural capacity of the cap beam (if present)
- $M_w$  – the flexural capacity of the railing about its vertical axis
- $M_c$  – the flexural capacity of the railing about a horizontal axis

LRFD Article 13.1 cautions designers that railings placed on retaining walls or spread footings may require investigation beyond that presented in this example. The governing or controlling yield line mechanism is assumed to form in the railing. If additional mechanisms with potentially lower load capacities are possible, designers should investigate them. The yield line mechanisms vary with rail location. Interior rail regions are assumed to have three yield lines. Two of the yield lines have tension on the inside of the railing and one has tension on the outside of the railing. See Figure 13.3.1.1, reproduced from LRFD Figure CA13.3.1-1.

The assumed failure mechanism at the end of rail sections (near deflection joints, expansion joints, openings, etc.) has one yield line that produces tension on the inside face of the railing. See Figure 13.3.1.2, reproduced from LRFD Figure CA13.3.1-2.



**Figure 13.3.1.1**  
**Yield Line Analysis for Interior Region**



**Figure 13.3.1.2**  
**Yield Line Analysis for End Region**

Figure 13.3.1.3 contains a rail elevation detail that identifies the location of interior and end regions. The length of end regions and interior regions is dependent on the relative flexural capacities of the railing ( $M_w$  and  $M_c$ ). The design example uses  $L_{ce}$  to represent the length of end regions and  $L_{ci}$  to represent the length of interior yield line mechanisms. Holding  $M_w$  constant, rail sections with larger  $M_c$  resistances have shorter and steeper yield line mechanisms.

Designers should note that in addition to inclined yield lines, one-way cantilever resistance of the rail should be investigated for rail segments with lengths less than twice  $L_{ce}$ .

### ***B. Barrier Flexural Resistance***

Three section details of a Type F barrier are presented in Figure 13.3.1.4. The top section presents typical reinforcement and geometry. The horizontal reinforcement consists of eight #13 bars. Two #16 bars are used for the vertical reinforcement. The R1601E bar is anchored in the deck and projects 10" into the rail. The R1602E bar is a closed stirrup that laps the R1601E bar.

The center detail in Figure 13.3.1.4 labels the horizontal reinforcement and identifies the "d" dimension assumed in  $M_w$  calculations. At any one yield line location four bars are assumed to provide flexural resistance and four bars are assumed available to carry shear loads via shear friction.

### ***[CA13.3.1]***

The bottom detail in Figure 13.3.1.4 identifies the "d" dimension of the vertical reinforcement at different locations. These values are averaged to compute  $M_c$ .

#### **Determine $M_b$**

The Type F barrier has no additional beam section at its top. Consequently, the  $M_b$  term is equal to zero in the rail resistance computations.

#### **Determine $M_w$**

Using the center detail of Figure 13.3.1.4 the flexural capacity about a vertical axis is computed. Bars 1, 3, 5, and 7 are assumed effective for yield lines that produce tension on the inside face of the rail. Bars 2, 4, 6, and 8 are assumed effective for the case where the yield line has tension on the outside face of the rail.

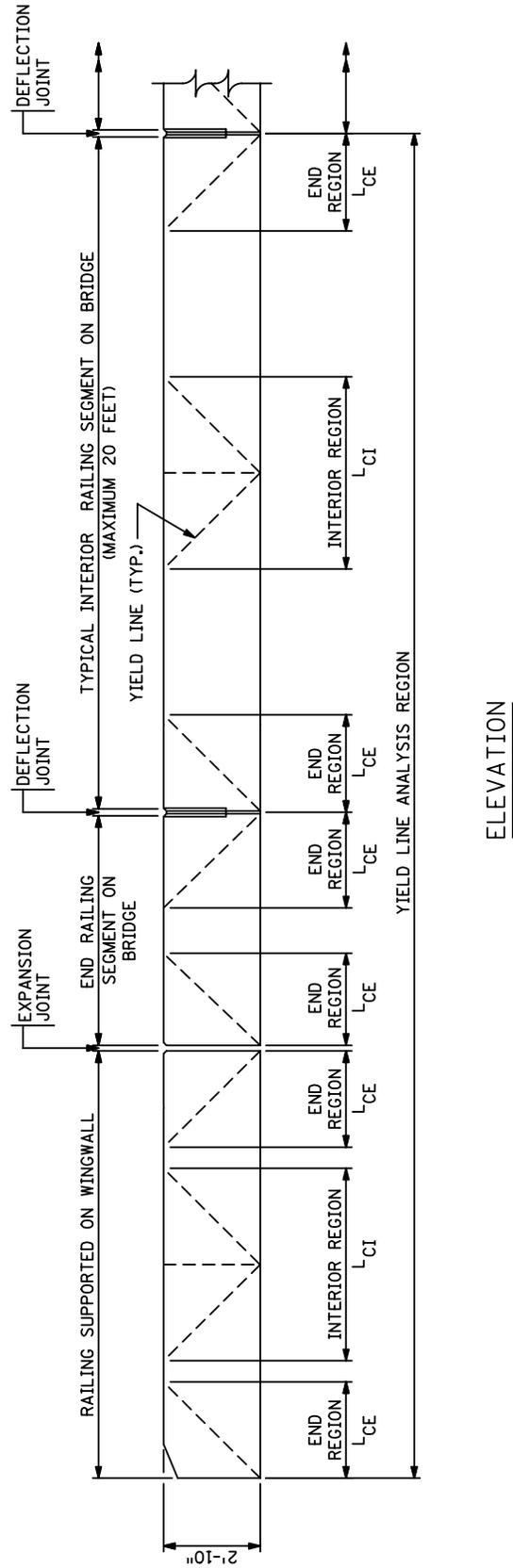
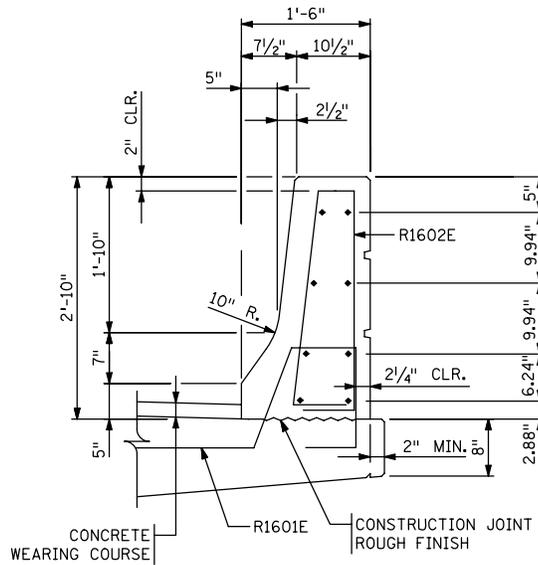
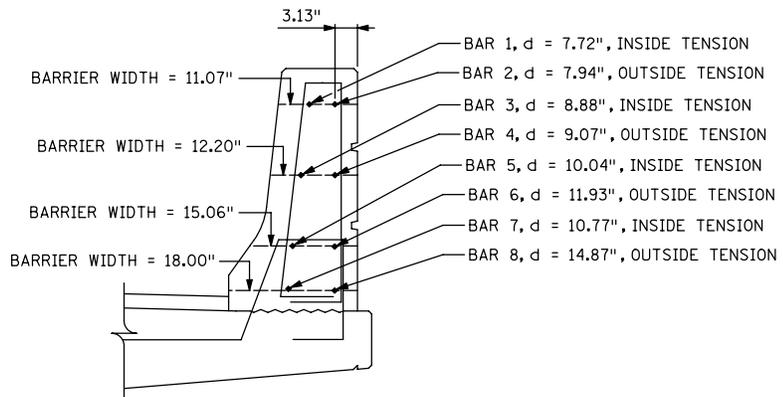


Figure 13.3.1.3

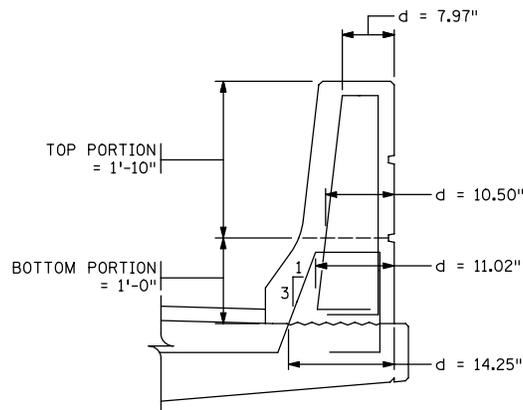


NOTE:  
FOR SIMPLICITY IN DETERMINING BAR LOCATIONS, SQUARE CORNERS ARE ASSUMED FOR BARS R1601E AND R1602E.

TYPICAL SECTION  
DERIVED FROM FIG. 5-397.117



SECTION FOR  $M_w$



SECTION FOR  $M_c$  & SHEAR

Figure 13.3.1.4

[5.7.3.2]

[1.3.2.1]

 **$M_w$  for Interior Region**

Capacities  $\phi M_n$  for a typical interior region are listed in the following table. The lever arm dimension of the different bars is found by subtracting half the depth of the flexural compression block.

$$\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right)$$

$$\phi = 1.0 \text{ (for Extreme Event Limit State)}$$

$$A_s = 0.20 \text{ in}^2$$

$$f_y = 60 \text{ ksi}$$

$$a = c \beta_1 = \frac{A_{s\text{total}} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{4 \cdot 0.20 \cdot 60}{0.85 \cdot 4.0 \cdot 34} = 0.42 \text{ in}$$

$$\frac{a}{2} = \frac{0.42}{2} = 0.21 \text{ in}$$

BAR	d (in)	Lever Arm $d - \frac{a}{2}$ (in)	$\phi M_{ni}$ for Inside Face Tension (k-in)	$\phi M_{no}$ for Outside Face Tension (k-in)
1	7.72	7.51	90.1	
2	7.94	7.73		92.8
3	8.88	8.67	104.0	
4	9.07	8.86		106.3
5	10.04	9.83	118.0	
6	11.93	11.72		140.6
7	10.77	10.56	126.7	
8	14.87	14.66		175.9
Totals			438.8	515.6

$$M_{wi} = \left( \frac{\phi M_{ni}}{H} \right) = \left( \frac{438.8 / 12}{2.83} \right) = 12.92 \text{ kip - ft/ft}$$

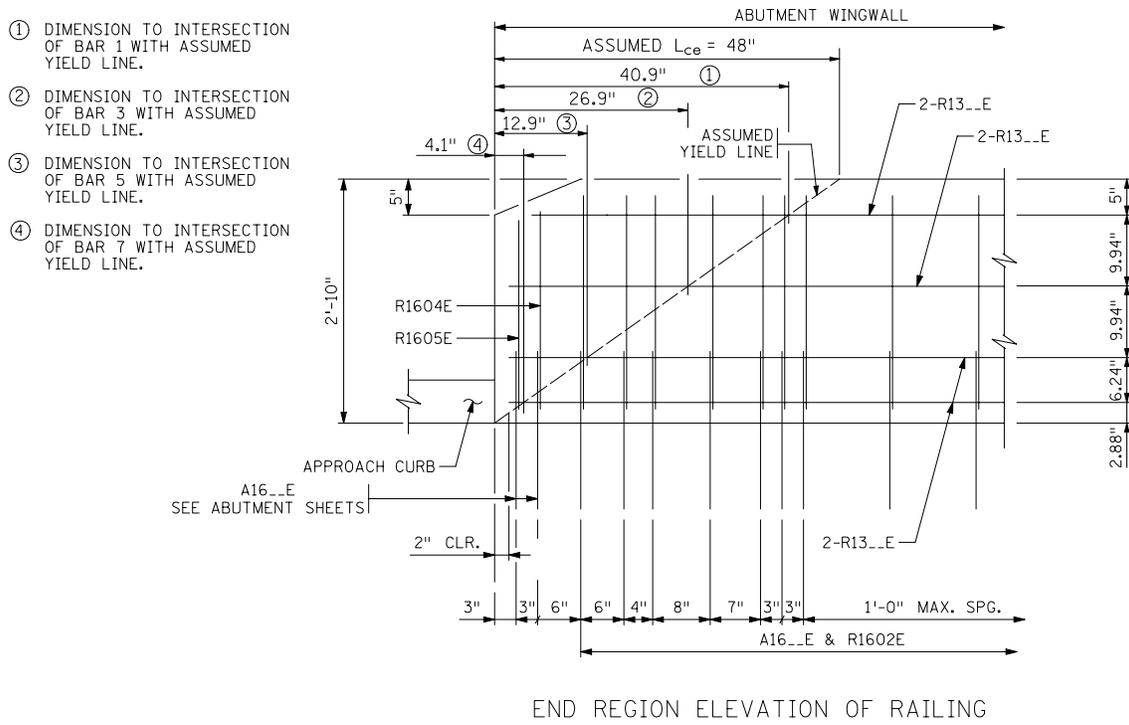
$$M_{wo} = \left( \frac{\phi M_{no}}{H} \right) = \left( \frac{515.6 / 12}{2.83} \right) = 15.18 \text{ kip - ft/ft}$$

For interior rail regions there is one outside tension yield line and two inside tension yield lines. Compute the average  $M_w$ :

$$M_{wint} = \frac{2 \cdot M_{wi} + 1 \cdot M_{wo}}{3} = \frac{2 \cdot 12.92 + 1 \cdot 15.18}{3} = 13.7 \text{ kip - ft/ft}$$

**M<sub>w</sub> for End Region**

At end regions not all of the horizontal bars will be fully developed by the time they intersect with the anticipated yield line. Assume the L<sub>ce</sub> dimension is at least 4.0 feet. The #13 bars have a development length of 12". Figure 13.3.1.5 shows the reinforcement in the end region of the rail in relation to the assumed yield line.



**Figure 13.3.1.5**

Similar to the interior region, the lever arm is found by subtracting off one half of the depth of the flexural compression block.

$$\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right)$$

$$\phi = 1.0 \text{ (for Extreme Event Limit State)}$$

$$A_s = 0.20 \text{ in}^2$$

$$f_y = 60 \text{ ksi}$$

$$a = c \beta_1 = \frac{A_{\text{total}} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.62 \cdot 60}{0.85 \cdot 4.0 \cdot 34} = 0.32 \text{ in}$$

$$\frac{a}{2} = \frac{0.32}{2} = 0.16 \text{ in}$$

Capacities  $\phi M_n$  for the end region are listed in the following table.

BAR	Embedded Length (in)	Bar Fraction Developed	Developed Bar Area $A_s$ (in. )	d (in)	Lever Arm $d - \frac{a}{2}$ (in)	$\phi M_n$ for Inside Face Tension (k-in)
1	36	1.00	0.20	7.72	7.56	90.7
3	24.9	1.00	0.20	8.88	8.72	104.6
5	10.9	0.91	0.18	10.04	9.88	106.7
7	2.1	0.18	0.04	10.77	10.61	25.5
		Total	0.62		Total	327.5

$M_w$  is found by averaging the capacity of the rail over the height of the rail.

$$M_{w\text{end}} = \left( \frac{\phi M_n}{H} \right) = \left( \frac{327.5 / 12}{2.83} \right) = 9.6 \text{ kip-ft/ft}$$

### Determine $M_c$

The Type F barrier does not have a uniform thickness. Consequently the "d" dimension of the vertical reinforcement varies with the vertical location in the rail. Averaged "d" dimensions are used to compute  $M_c$  separately for the top and bottom sections. Then a weighted average of the two sections is taken to determine  $M_c$  for the entire rail section. Using "d" dimensions labeled in the bottom detail of Figure 13.3.1.4, the average "d" dimensions can be found.

Location	d (in)	Average d (in)
Top	7.97	9.24
Mid Top	10.50	
Mid Bottom	11.02	12.64
Bottom	14.25	

### $M_c$ for Interior Region

The internal flexural lever arm is dependent on the amount of reinforcement in the cross section. The maximum spacing of vertical steel in interior regions is 12". Use a 12" vertical steel spacing to evaluate the interior rail region.

For the top portion,  $A_{\text{stop}} = 0.31 \text{ in}^2/\text{ft}$

$$a_{\text{top}} = c \beta_1 = \frac{A_{\text{stop}} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.31 \cdot 60}{0.85 \cdot 4.0 \cdot 12.0} = 0.46 \text{ in}$$

$$M_{ctop} = \phi M_n = \phi A_{stop} f_y \left( d_{top} - \frac{a_{top}}{2} \right) = 1.0(0.31)(60) \cdot \left( 9.24 - \frac{0.46}{2} \right) \cdot \left( \frac{1}{12} \right)$$

$$= 14.0 \text{ kip-ft/ft}$$

For the bottom portion, the R1601E bars are not fully developed at the rail/deck interface. Determine bar development fraction:

For a straight #16 bar, the basic development length  $\ell_{db}$  is:

$$\ell_{db} = \frac{1.25 A_b f_y}{\sqrt{f'_c}} = \frac{1.25(0.31)(60)}{\sqrt{4}} = 11.63 \text{ in}$$

or

$$\ell_{db} = 0.4 d_b f_y = 0.4(0.625)(60) = 15.00 \text{ in} \quad \underline{\text{GOVERNS}}$$

Using modification factors for epoxy coating (1.2) and bar spacing  $> 6''$  with  $> 3''$  cover (0.8), the straight bar development length is:

$$\ell_{db} = 1.2(0.8)(1500) = 14.40 \text{ in}$$

For a hooked #16 bar, the basic development length  $\ell_{hb}$  is:

$$\ell_{hb} = \frac{38.0 \cdot d_b}{\sqrt{f'_c}} = \frac{38.0(0.625)}{\sqrt{4}} = 11.88 \text{ in}$$

Using modification factors for epoxy coating (1.2) and cover (0.7), the hooked bar development length is:

$$\ell_{dh} = 1.2(0.7)(11.88) = 9.98 \text{ in}$$

Therefore, the benefit derived from the hook is:

$$14.40 - 9.98 = 4.42 \text{ in}$$

The R1601E bar is hooked with a vertical embedment of 5.18 in.

Then the development fraction is:

$$F_{dev} = \frac{5.18 + 4.42}{14.40} = 0.67$$

The required extension beyond the 90° bend for a standard hook (A or G dimension) is 10" for a #16 bar. The R1601E bar has an extension of 18". Because of this extra extension and the fact that the 18" extension will have to pull through the top mat of reinforcement in order for the bar to fail, assume a higher development fraction  $F_{dev} = 0.75$ .

Then  $A_{sbot} = 0.75(0.31) = 0.23 \text{ in}^2/\text{ft}$

$$a_{bot} = c\beta_1 = \frac{A_{sbot} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.23 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.34 \text{ in}$$

$$\begin{aligned} M_{cbot} &= \phi M_n = \phi A_{sbot} f_y \left( d_{bot} - \frac{a_{bot}}{2} \right) \\ &= 1.0(0.23)(60) \left( 12.64 - \frac{0.34}{2} \right) \left( \frac{1}{12} \right) = 14.3 \text{ kip-ft/ft} \end{aligned}$$

$$M_{cint} = \frac{14.0(1.83) + 14.3(1.00)}{2.83} = 14.1 \text{ kip-ft/ft}$$

### $M_c$ for End Region

The end region has nine A16 and nine R16 bars in the end 4.0 feet of the rail. For the last R16 bar, due to the small amount of bar extending above the yield line, consider only 8 bars to be effective in resisting load.

Then, the average  $A_{stop} = 0.62 \text{ in}^2/\text{ft}$

$$a_{top} = c\beta_1 = \frac{A_{stop} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.62 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.91 \text{ in}$$

$$\begin{aligned} M_{ctop} &= \phi M_n = \phi A_{stop} f_y \cdot \left( d_{top} - \frac{a_{top}}{2} \right) = 1.0(0.62)(60) \cdot \left( 9.24 - \frac{0.91}{2} \right) \left( \frac{1}{12} \right) \\ &= 27.2 \text{ kip-ft/ft} \end{aligned}$$

The average effective  $A_{sbot} = 0.75(0.62) = 0.47 \text{ in}^2/\text{ft}$

$$a_{bot} = c\beta_1 = \frac{A_{sbot} \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{0.47 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.69 \text{ in}$$

$$\begin{aligned} M_{cbot} &= \phi M_n = \phi A_{sbot} f_y \cdot \left( d_{bot} - \frac{a_{bot}}{2} \right) = 1.0(0.47)(60) \cdot \left( 12.64 - \frac{0.69}{2} \right) \left( \frac{1}{12} \right) \\ &= 28.9 \text{ kip-ft/ft} \end{aligned}$$

$$\text{Then } M_{cend} = \frac{27.2(1.83) + 28.9(1.00)}{2.83} = 27.8 \text{ kip-ft/ft}$$

**C. Flexural  
Capacity Check**

With  $M_w$  and  $M_c$  computed for an interior and end region, the resistance of the railing can be computed with the equations in LRFD Article A13.3.1.

**Check the Capacity of an Interior Region**

With  $M_{bint} = 0$ ,  $M_{wint} = 13.7$  kip-ft/ft and  $M_{cint} = 14.1$  kip-ft/ft, the length of the yield line mechanism and the resistance of the mechanism can be found:

$$[Eqn A13.3.1-1] \quad L_{ci} = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \left[\frac{8 \cdot H \cdot (M_{bint} + M_{wint} \cdot H)}{M_{cint}}\right]} = 9.8 \text{ ft}$$

$$[Eqn A13.3.1-2] \quad R_{wi} = \left(\frac{2}{2 \cdot L_{ci} - L_t}\right) \left(8 \cdot M_{bint} + 8 \cdot M_{wint} \cdot H + \frac{M_{cint} \cdot L_{ci}^2}{H}\right) = 98.0 \text{ kips}$$

which, is greater than the 54 kip extreme event design load.

**Check the Capacity of the End Region**

With  $M_{bend} = 0$ ,  $M_{wend} = 9.6$  kip-ft/ft and  $M_{cend} = 27.8$  kip-ft/ft, the length of the yield line mechanism and the resistance of the mechanism can be found:

$$Eqn A13.3.1-4 \quad L_{ce} = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H \cdot \left(\frac{M_{bend} + M_{wend} \cdot H}{M_{cend}}\right)} = 4.2 \text{ ft}$$

$$[Eqn A13.3.1-3] \quad R_{we} = \left(\frac{2}{2 \cdot L_{ce} - L_t}\right) \left(M_{bend} + M_{wend} \cdot H + \frac{M_{cend} \cdot L_{ce}^2}{H}\right) = 81.8 \text{ kips}$$

which, is also greater than the required load capacity of 54 kips.

The other end regions are to be checked similarly.

**D. Shear Capacity  
Check**

Use shear friction methods to evaluate the shear capacity of the joint between the deck and railing. Assume that  $F_t$  and  $F_L$  occur simultaneously.

The resultant shear force is:

$$V_{res} = \sqrt{F_t^2 + F_L^2} = \sqrt{54^2 + 18^2} = 56.9 \text{ kips}$$

**[5.8.4]**

The basic shear capacity equation for a section using shear friction is:

$$\phi V_n = \phi \cdot [C \cdot A_{cv} + \mu (A_{vf} \cdot f_y + P_c)]$$

Neglect cohesion and the small permanent compression across the interface due to selfweight. Conservatively assume that the interface between the railing and the deck is not roughened. The appropriate friction factor  $\mu$  is 0.60.

Substitute  $V_{res}$  for  $\phi V_n$  rearranging the remaining terms, and solve for the required area of reinforcement:

$$A_{vfreq} = \left( \frac{V_{res}}{\phi_v \cdot \mu \cdot f_y} \right) = \left( \frac{56.9}{1.0 \cdot 0.60 \cdot 60} \right) = 1.58 \text{ in}^2$$

The required number of #16 bar legs is:

$$\left( \frac{A_{vfreq}}{A_b} \right) \left( \frac{1.58}{0.31} \right) = 5.1 \text{ legs}$$

Check the interior region first. Assuming the #16 bars are at the maximum spacing of 12" and the  $L_{ci}$  dimension is 9.9 feet, 10 bars will be provided.

At the end region, nine #16 bars are provided in the end 4.2 feet ( $L_{ce}$ ). Both interior and end regions have adequate shear capacity at the deck railing interface.

### E. Summary

When checked in accordance with the procedure shown within this example, the capacity of the end regions adjacent to the expansion joint and deflection joints did not meet the required 54 kip load capacity.

Because the neutral axis is located very close to the outside face of the rail for determination of both  $M_w$  and  $M_c$ , all of the regions were reanalyzed to take advantage of the additional capacity provided by the outside face reinforcement. Therefore, in the second analysis, both the inside face rail reinforcement and the outside face rail reinforcement were included in the determination of the rail capacity. The revised values for the F-rail are:

Interior Region:

With wearing course

$$L_{ci} = 10.2 \text{ ft}$$

$$R_{wi} = 122.9 \text{ kip}$$

Without wearing course

$$L_{ci} = 9.9 \text{ ft}$$

$$R_{wi} = 124.1 \text{ kip}$$

End Region:

With wearing course

$$L_{ce} = 4.6 \text{ ft}$$

$$R_{we} = 57.2 \text{ kip}$$

Without wearing course

$$L_{ce} = 4.6 \text{ ft}$$

$$R_{we} = 59.2 \text{ kip}$$

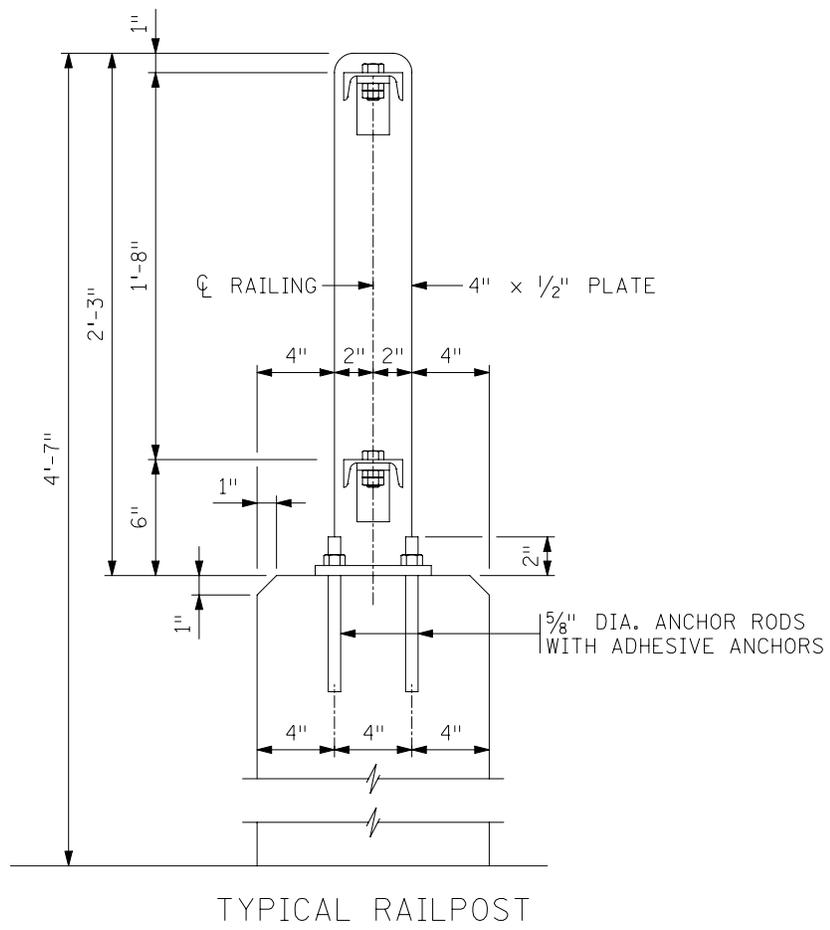
Adequate vehicle collision load capacity is provided with the default reinforcing provided in Bridge Details Part II Figure 5-397.117. (See Figure 13.3.1.6.)



**13.3.2 Adhesive Anchor Design Example**

The objective of this example is to design adhesive anchors (as an alternate to the cast-in-place anchorage) to secure a metal railing atop a concrete barrier. The railing under consideration is Mn/DOT 5-397.154 "Metal Railing for Bikeways (Type M-1) and Concrete Parapet (Type P-1) (with Integral End Post)". The standard anchorage elements beneath each vertical post are four cast-in-place 5/8" x 8" anchor bolts. All steel components for the railing have a yield strength of 36 ksi. The concrete used for the parapet has a design compressive strength of 4 ksi.

The example is structured in a top-down fashion. After determining the design loads, the railpost and base plate are checked. After that, the shear and tensile capacity of the anchors is computed. For each of the forces, the resistance of steel and concrete is determined individually.



**Figure 13.3.2.1**

Reference material on the design of adhesive anchors is limited. The equations for concrete shear capacity and tension capacity, and modifiers for group effect and for edge effect presented in this example are based on material in *Behavior and Design of Fastening to Concrete*, Richard E. Klingner, University of Texas at Austin, 48<sup>th</sup> Annual Concrete Conference, University of Minnesota, December 3, 1998 and *ACI 318, Appendix D*. Reference material on the design of non-adhesive anchors can also be found in Chapter 6 of the *PCI Design Handbook*.

Figure 13.3.2.1 presents the typical railpost detail for the railing. The maximum distance L between railposts is 10'-0".

### A. Design Loads

Section 13 of the LRFD Specifications covers bridge railings. Article 13.8.2 lists the loads to consider for the design of rail elements and posts for pedestrian and bicycle railings. Design railposts to resist concentrated design live load  $P_{LL}$  applied at the height of the top rail element.

#### [Eqn 13.8.2-1]

$$P_{LL} = 0.20 + 0.050 \cdot L = 0.20 + 0.050 \cdot 10 = 0.70 \text{ kips}$$

#### [Table 3.4.1-1]

Using a load factor of 1.75 for live load results in a design horizontal force of:

$$H_u = 1.75 \cdot P_{LL} = 1.75 \cdot 0.70 = 1.23 \text{ kips}$$

Per Figure 13.3.2.1, the lever arm from top rail to top of concrete is 2.17 feet. The design moment at the bottom of the base plate is:

$$M_{upost} = H_u \cdot d = (1.23) \cdot (2.17) = 2.66 \text{ k-ft} = 31.9 \text{ kip-in}$$

### B. Railpost Design Check

Begin by checking the railpost. It must have adequate capacity to resist the design moment. By inspection, the rail elements provide adequate bracing to develop the yield moment of the section. Therefore, the capacity is:

#### [6.12.2.2.4b]

$$M_n = M_y = F_y \cdot S$$

The railpost is a  $1/2$ " x 4" plate loaded about its strong axis.

$$S_{post} = \frac{b \cdot d^2}{6} = \frac{0.5 \cdot 4^2}{6} = 1.33 \text{ in}^3$$

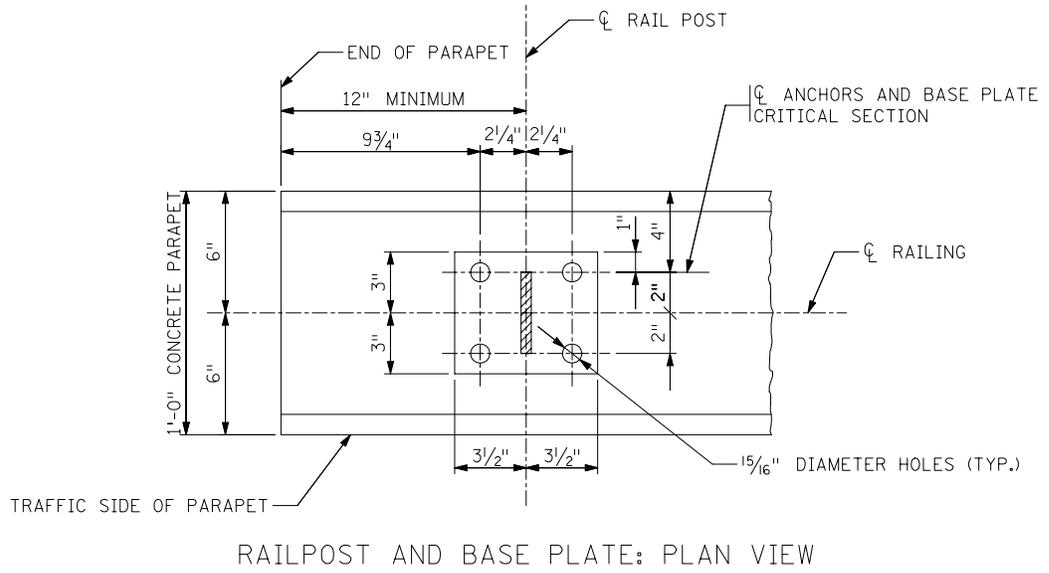
[6.5.4.2]

For steel elements in flexure,  $\phi_f = 1.00$ .

$$\phi_f \cdot M_n = \phi_f \cdot S_{post} \cdot F_y = 1.00 \cdot 1.33 \cdot 36 = 48 \text{ kip-in} > 31.9 \text{ kip-in} \quad \text{OK}$$

**C. Base Plate Design Check**

A plan view of the base plate is shown in Figure 13.3.2.2. Assume that the critical section occurs at the face of the vertical post (1" from the edge of the plate on the compression side).



**Figure 13.3.2.2**

Conservatively assume that the compression reaction,  $R_{comp}$ , acts at the edge of the base plate. The internal lever arm between the anchors and the compression edge of the plate is 5". Then,

$$R_{comp} = \frac{M_{upost}}{arm_{int}} = \frac{31.9}{5.0} = 6.38 \text{ kips}$$

and

$$M_{uplate} = R_{comp} \cdot arm_{plate} = 6.38 \cdot 1.0 = 6.38 \text{ kip-in}$$

The resisting moment at the face of the column is the capacity of the plate minus two anchor bolt holes.

$$S_{plate} = \frac{(b_{plate} - 2 \cdot d_{hole}) \cdot t_{plate}^2}{6} = \frac{(7 - 2 \cdot 0.9375) \cdot 0.5^2}{6} = 0.214 \text{ in}^3$$

$$\begin{aligned} M_{rplate} &= \phi_f \cdot S_{plate} \cdot F_y = 1.00 \cdot 0.214 \cdot 36 \\ &= 7.70 \text{ kip-in} > 6.38 \text{ kip-in} \quad \text{OK} \end{aligned}$$

**D. Adhesive  
Anchor Design  
Forces**

**Factored Shear Force**

Assume that the base plate engages each of the anchors equally. Then,

$$V_u = \frac{H_u}{4} = \frac{1.23}{4} = 0.31 \text{ kips/anchor}$$

**Factored Tension Force**

Determine the factored tension load  $T_u$  on one anchor using the overturning moment  $M_{u\text{post}}$  (31.9 kip-in). As a simplifying design practice Mn/DOT uses the distance between anchor rods (4.0 in) as the flexural lever arm:

$$T_u = \frac{M_u}{\text{arm} \cdot N} = \frac{31.9}{(4.0) \cdot 2} = 3.99 \text{ kips/anchor}$$

**E. Anchor Rod  
Shear Capacity**

The anchor rods are assumed to have sufficient embedment to develop their shear capacity.

Try Mn/DOT 3385, Type A anchor rods.

$$F_y = 36 \text{ ksi and } F_{ub} = 58 \text{ ksi}$$

**[6.5.4.2]**

Since  $F_y$  of the Type A anchor rods is equal to  $F_y$  for A307 bolts, use  $\phi_s = 0.65$ .

Each anchor rod will be subject to one shear plane. Assume that threads are included in the shear plane. The area  $A_b$  of one  $5/8$ " diameter anchor rod is  $0.31 \text{ in}^2$ . Then,

**[6.13.2.7]**

$$R_n = 0.38 \cdot A_b \cdot F_{ub} \cdot N_s = 0.38 \cdot 0.31 \cdot 58 \cdot 1 = 6.83 \text{ kips}$$

$$\phi_s R_n = 0.65 \cdot 6.83 = 4.44 \text{ kips} > 0.31 \text{ kips} \quad \underline{OK}$$

**F. Concrete Shear  
Capacity**

The concrete shear capacity is a function of geometry and compressive strength. Assume the two anchors on the compression side of the base plate connection are the critical shear anchors. For calculation of shear capacity, consider "end effects", "edge effects", and "group effects". For this example, end effects need to be considered near the expansion joint and deflection joints in the parapet. Consider group effects based on the distances between anchors in a group. Widely spaced anchors function as individual anchors, while more closely spaced anchors have a reduced capacity.

For shear, the end effects, edge effects, and group effects are incorporated in the calculation for the concrete area effective in resisting shear. See Figure 13.3.2.3.

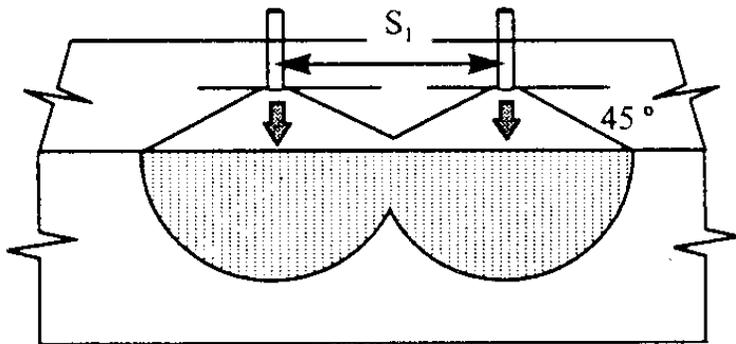
Per Mn/DOT policy the center of a railpost can be no closer than 12" to a deflection joint or an expansion joint end of the parapet. The anchors are located 2.25 inches away from the center of the railpost. Consequently, the end distance is  $d_{end} = 12 - 2.25 = 9.75$  in.

**[Klingner]**

The anchor rod edge distance  $c_1 = 4$  in. The influence distance for shear is:

$$1.5 \cdot c_1 = 1.5 \cdot 4.0 = 6.0 \text{ in} < 9.75 \text{ in}$$

Therefore, end effects need not be considered for shear.



$$A_v = \left( \pi - \frac{\pi\theta/2}{180} + \sin \theta \right) c_1^2$$

$$\theta = 2 \cos^{-1} \left( \frac{s_1}{2c_1} \right)$$

**Figure 13.3.2.3**  
**Two Anchor Shear Interface Area**  
**(From Klingner)**

Plugging values into the formula results in:

$$s_1 = 4.5 \text{ in}, c_1 = 4 \text{ in}$$

$$\theta = 2 \cdot \text{acos} \left( \frac{s_1}{2 \cdot c_1} \right) = 2 \cdot \text{acos} \left( \frac{4.5}{2 \cdot 4} \right) = 111.5^\circ$$

$$A_v = \left[ \pi - \frac{\pi \cdot \theta}{180} + \sin(\theta) \right] \cdot c_1^2 = 49.6 \text{ in}^2$$

The capacity of the concrete on this interface is:

**[Klingner]**

$$V_{c\_interface} = 4 \cdot \sqrt{f'_c} = \frac{4 \cdot \sqrt{4000}}{1000} = 0.253 \text{ ksi}$$

Concrete capacity of two anchors in shear is:

$$V_{c2} = A_v \cdot V_{c\_interface} = (49.6) \cdot (0.253) = 12.54 \text{ kips}$$

Concrete capacity of one anchor in shear is:

$$V_{c1} = \frac{V_{c2}}{2} = \frac{12.54}{2} = 6.27 \text{ kips}$$

$$\phi V_{c1} = 0.90 \cdot 6.27 = 5.64 \text{ kips} > 0.31 \text{ kips} \quad \underline{\text{OK}}$$

**[Klingner]**

Because the shear demand is less than 20% of the shear capacity, ignore the interaction effects between shear and tension.

### **G. Anchor Rod Tension Capacity**

Determine the capacity of the anchor rods. Begin by checking if interaction effects need to be considered.

**[6.13.2.11]**

$$\frac{P_u}{R_n} = \frac{0.31}{6.83} = 0.045 \leq 0.33$$

The tension capacity can be found without considering shear. The tension capacity of each anchor rod is:

$$T_n = 0.76 \cdot A_b \cdot F_{ub} = (0.76) \cdot (0.31) \cdot (58) = 13.66 \text{ kips}$$

**[6.5.4.2]**

Using  $\phi_t$  for an A307 bolt ( $\phi_t = 0.80$ ),

$$\phi T_n = 0.80 \cdot (13.66) = 10.93 \text{ kips} > 3.99 \text{ kips} \quad \underline{\text{OK}}$$

### **H. Resistance Factor for Adhesive Anchor Pullout**

In the past, adhesive anchors were designed with allowable stress methods. A typical factor-of-safety (FS) was 4. A similar safety or reliability level will be used for LRFD designs. The load factor for live loads is 1.75. Choose a resistance factor that when combined with the load factor for live load will produce a factor near 4.

$$\text{If } FS = \frac{\gamma_{LL}}{\phi_a}, \text{ then } \phi_a = \frac{\gamma_u}{FS} = \frac{1.75}{4} \quad \text{Use } \phi_a = 0.45$$

### ***I. Pullout Capacity of Adhesive Anchor***

According to research referenced by Klingner, the best model for tensile behavior of adhesive anchors is a simple bond model that assumes a uniform bond stress over the length of the anchor. Taking into account end effects, edge effects, and group effects, the factored tensile resistance  $\phi_a T_{na}$  is:

$$\phi_a \cdot T_{na} = \phi_a \cdot T_{n0} \cdot \psi_c \cdot \psi_e \cdot \psi_g$$

where,  $T_{n0}$  = nominal adhesive tensile capacity =  $\tau_{\text{bond}} \cdot (\pi \cdot d_{\text{anchor}} \cdot L_{\text{conc}})$

$\tau_{\text{bond}}$  = ultimate bond stress of adhesive

$d_{\text{anchor}}$  = diameter of steel anchor

$L_{\text{conc}}$  = steel anchor embedment length

$\psi_c$  = concrete strength variation factor

$\psi_e$  = end/edge effect factor

$\psi_g$  = group effect factor

Based on adhesive anchor product literature for a  $\frac{5}{8}$ " diameter threaded rod anchored in concrete with  $f'_c = 4$  ksi, use an ultimate bond stress  $\tau_{\text{bond}} = 2$  ksi for the adhesive.

The concrete strength variation factor  $\psi_c$  accounts for variations in bond stress with changes in concrete strength. For concrete strengths greater than 3 ksi,  $\psi_c$  can conservatively be taken equal to 1.0.

#### **End Effect and Edge Effect**

The end/edge effect correction is independent of the depth of embedment. It is only dependent on the ratio of the end/edge distance to the diameter of the anchor. Consider end/edge effects when adhesive anchors are located within 10 anchor diameters of an edge.

$$c_0 = 10 \cdot d_{\text{anchor}} = 10 \cdot 0.625 = 6.25 \text{ in}$$

$$\text{Actual edge distance } c_1 = 4.0 \text{ in} < 6.25 \text{ in}$$

$$\text{then, } \psi_e = \frac{0.4}{10} \cdot \left( \frac{c_1}{d_{\text{anchor}}} \right) + 0.60 = \frac{0.4}{10} \cdot \left( \frac{4}{0.625} \right) + 0.60 = 0.86$$

***[Klingner]***

#### **Group Effect**

The reduction in capacity due to group effects is a ratio of the sum of influence areas for single anchors to that of the group. It is dependent on the depth of embedment and the spacing between anchors. The minimum embedment length  $h_{\text{emin}}$  for an adhesive anchor is  $6 \cdot d_{\text{anchor}}$ :

$$h_{\text{emin}} = 6 \cdot d_{\text{anchor}} = 6 \cdot 0.625 = 3.75 \text{ in}$$

Try an embedment length  $h_e = 4.0$  in

**[Klingner]**

The critical spacing between anchors,  $s_o$ , where group effects disappear is:

$$s_o = 1.75 \cdot h_e = 7.0 \text{ in}$$

The actual spacing between anchors ( $s_1$  dimension) is 4.5 inches. Therefore, use a group effect reduction in capacity.

Figure 13.3.2.4 shows the influence area for anchors with an embedment of 4".

**[Klingner]**

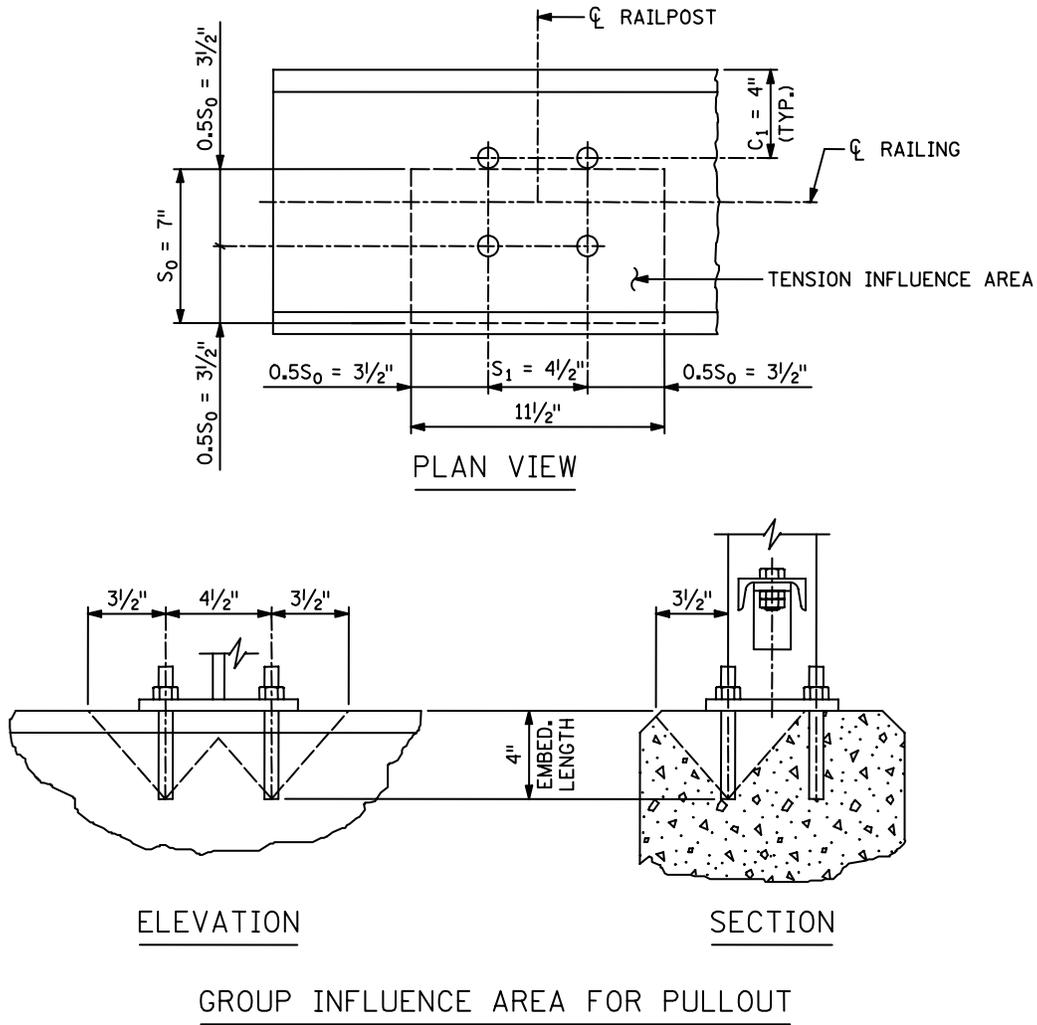
The influence area of a single anchor is:

$$A_o = 3 \cdot h_e^2 = 3 \cdot (4.0)^2 = 48 \text{ in}^2$$

The influence area of two anchors with  $s_1$  equal to 4.5 inches is:

$$A_{n2} = s_o \cdot (s_o + s_1) = 7.0 \cdot (7.0 + 4.5) = 80.5 \text{ in}^2$$

$$\Psi_g = \frac{A_{n2}}{2 \cdot A_o} = \frac{80.5}{2 \cdot 48} = 0.84$$



**Figure 13.3.2.4**

**Pullout Capacity and Embedment**

During construction, the contractor will select a Mn/DOT Approved Concrete Anchorage, which are listed at the Mn/DOT Office of Materials Web site (<http://www.mrr.dot.state.mn.us>). The approved product must have a factored tensile resistance  $\phi \cdot T_{na}$  that is at least equal to the factored tension force  $T_u$  determined in design:

$$\phi_a \cdot T_{na} = \phi_a \cdot T_{n0} \cdot \psi_c \cdot \psi_e \cdot \psi_g \geq T_u$$

$$\text{then, } T_{n0} \geq \frac{T_u}{\phi_a \cdot \psi_c \cdot \psi_e \cdot \psi_g} = \frac{3.99}{0.45 \cdot 1.0 \cdot 0.86 \cdot 0.84} = 12.3 \text{ kips}$$

Assuming  $1/2$ " of top surface deterioration, the minimum anchor embedment depth  $L_{\text{conc}}$  is:

$$L_{\text{conc}} \geq \left( \frac{T_{n0}}{\tau_{\text{bond}} \cdot \pi \cdot d_{\text{anchor}}} \right) + 0.5 \text{ in}$$

$$= \left( \frac{12.3}{2 \cdot \pi \cdot 0.625} \right) + 0.5 = 3.63$$

This is less than the assumed 4". OK

At the job site, anchors are subjected to a proof load test. The proof load will be the smaller of:

A limit based on yielding the steel rod:

$$\frac{2}{3} \cdot (A_b \cdot F_y) = \frac{2}{3} \cdot (0.31 \cdot 36) = \frac{2}{3} \cdot (11.16) = 7.4 \text{ kips}$$

A limit based on the nominal adhesive capacity:

$$\frac{1}{2} \cdot (T_{n0}) = \frac{1}{2} \cdot (12.3) = 6.2 \text{ kips} \quad \text{GOVERNS}$$

#### **J. Summary**

An adhesive anchor detail with the following properties has adequate capacity to support the Type M-1 railing: The anchor rods shall be  $5/8$ " diameter, Mn/DOT 3385 Type A anchor rods with a 4" minimum embedment. The adhesive shall have a minimum ultimate pull-out strength of 12.3 kips. The proof load for field testing shall be 6.2 kips.

#### **K. Adhesive Anchor Design for Traffic Rails**

The design of adhesive anchors for traffic rails is different than the design of adhesive anchors for pedestrian rails shown above. A traffic rail requires reinforcement or anchor rods to withstand a vehicle crash load under the Extreme Event II limit state. For a metal rail on parapet system or a concrete barrier where the design is based on successful crash testing along with a yield line analysis, design the adhesive to develop the strength of the reinforcement bar or anchor rod.

The Extreme Event II limit state has a load factor of 1.0 for the vehicle crash load. Using the procedure in Article 13.3.2H of this manual to determine a resistance factor results in the following:

$$\phi_a = \frac{\gamma_{\text{CT}}}{\text{FS}} = \frac{1.0}{4} = 0.25 \quad (\text{This value seems very low.})$$

The factor of safety (FS) of 4 used by adhesive manufacturers is based on a working load and not an extreme event load. Using the low crash probability under an extreme event and the non-working load nature of the crash load as a basis, Mn/DOT policy is to design for an FS of 1.66.

Then,

$$\phi_a = \frac{\gamma_{CT}}{FS} = \frac{1.0}{1.66} = 0.60 \quad (\text{Use } \phi_a = 0.60 \text{ for traffic rail only.})$$

Consider the following example:

A rail reconstruction project requires the use of #16 bars @ 12" spacing to anchor a new F-rail to an existing deck with adhesive anchors.

$$T_u = A_s \cdot F_y = 0.31 \cdot 60 = 18.6 \text{ kips}$$

$$\phi_a \cdot T_{na} = \phi_a \cdot T_{n0} \cdot \psi_c \cdot \psi_e \cdot \psi_g \geq T_u$$

Assuming that  $\psi_c \cdot \psi_e \cdot \psi_g = 1.0$ :

$$T_{n0} \geq \frac{T_u}{\phi_a} = \frac{18.6}{0.60} = 31.0 \text{ kips}$$

Based on adhesive anchor product literature for a #16 bar anchored in concrete with  $f'_c = 4$  ksi, use an ultimate bond stress  $\tau_{\text{bond}} = 2.5$  ksi. Assuming  $1/2$ " of top surface deterioration, the minimum required embedment is:

$$\begin{aligned} L_{\text{conc}} &= \left( \frac{T_{n0}}{\tau_{\text{bond}} \cdot \pi \cdot d_{\text{anchor}}} \right) + 0.5 \\ &= \left( \frac{31.0}{2.5 \cdot \pi \cdot 0.625} \right) + 0.5 = 6.82 \text{ in} \quad \underline{\text{Say 7" min. embedment}} \end{aligned}$$

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**14. JOINTS AND BEARINGS**

Expansion joints and bearings provide mechanisms to accommodate movements of bridges without generating excessive internal forces. This section provides guidance on joint and bearing selection and the movement and loads that must be used in their designs.

**14.1 Bridge Movements and Fixity**

To determine movements for bearings and joints, the point of fixity must be established for the bridge or bridge segment. The point of fixity is the neutral point on the bridge that does not move horizontally as the bridge experiences temperature changes. Use the following guidance concerning bridge fixity:

- 1) For single span structures, fix the bearings at the low end of the bridge.
- 2) For typical two-span structures, fix the bearings at the pier. For bridges with tall or flexible piers that are located on slopes, the superstructure may tend to move toward the downhill end, causing maintenance problems at the uphill end due to a wider joint than anticipated. For these bridges, consider providing fixed bearings at the downhill abutment.
- 3) For structures with three or more spans, investigate the longitudinal stiffness of the bridge. The longitudinal stiffness is a function of the interaction between pier stiffnesses, bearing types and joint locations. Consider the following:
  - a) The number and location of expansion joints is determined based on a maximum joint opening of 4 inches at the ends of the bridge. When joint openings exceed 4 inches, two options are available:
    - i) The preferred option is to provide additional joints at the piers to split the superstructure into segments.
    - ii) Provide modular expansion joints at bridge ends only.
  - b) For each bridge or bridge segment, provide fixed bearings at a minimum of two piers to provide increased resistance to longitudinal movements.
  - c) Provide fixed bearings at all tall pier locations. Tall or flexible piers deflect prior to mobilizing the translational capacity of the bearing.
  - d) Bridges with tall or flexible piers that are located on slopes may tend to move toward the downhill end, causing maintenance problems at the uphill end due to a wider joint than anticipated. For these bridges, consider providing fixed bearings at the downhill abutment.
  - e) A combination of fixed, expansion, guided, and limited expansion bearings can be provided at the piers to accommodate the movements for the bridge or bridge segments.

- f) Based on the point of fixity of each segment, the maximum movements can be determined for the design of joints and bearings.

**14.2 Expansion Joints [14.5.3.2]**

Minnesota bridges with parapet type abutments typically have strip seal expansion joints at the abutments to isolate superstructure movements from the abutments. When the maximum joint openings at the abutments exceed 4 inches additional joints are needed at piers or modular joints are required at the abutments.

Do not use elastomeric compression seal expansion joints.

**14.2.1 Thermal Movements [Table 3.4.1-1]**

Design joint openings for movements associated with a temperature range of 150°F (-30°F to 120°F). For strip seal expansion joints on typical bridges, use a load factor for movement of 1.0. (Note that this value differs from the LRFD Specification based on past performance of joints in Minnesota.) For strip seal expansion joints on non-typical bridges and for all modular expansion joints, use a load factor for movement of 1.2 per LRFD Article 3.4.1. See BDM Article 3.10.1 for the definition of typical and non-typical bridges.

**[5.4.2.2]  
[6.4.1]**

The coefficients of thermal expansion are:

- Concrete:  $6.0 \times 10^{-6}$  per °F
- Steel:  $6.5 \times 10^{-6}$  per °F

**14.2.2 Strip Seal Expansion Joints**

For movements of up to 4 inches, use strip seal expansion devices. Design joints to have a minimum opening of  $\frac{1}{2}$  inch between the steel elements (extrusions) of the joint.

**[14.5.3.2]**

To provide a reasonably smooth roadway surface, the maximum width of expansion openings is limited to 4 inches (measured perpendicular to joint) on roadway bridges. The maximum width for pedestrian bridges is 5 inches. Detail cover plates on sidewalks, medians, and pedestrian bridges to cover the opening.

The standard strip seal device is a Type 4.0, which has a movement capacity of 4 inches. Bridges on a horizontal curve or with a skew over 30° must accommodate "racking" or transverse movements as well. For these situations use a Type 5.0 strip seal (5 inch capacity). Type 5.0 strip seals can also be used on pedestrian bridges.

For skews less than 30°:

- For expansion distance less than 150'-0", dimension opening at 2 inches at all temperatures.
- For expansion distance greater than or equal to 150'-0", dimension opening at 1½ inches at 90°F. Also determine and show dimension at 45°F, checking that the opening at -30°F does not exceed 4 inches. If so, reduce accordingly at 45°F and 90°F.

For skews greater than or equal to 30°:

- Dimension opening at 1½ inches at 90°F. Also determine and show dimension at 45°F, checking that the opening at -30°F does not exceed 4 inches. If so, reduce accordingly at 45°F and 90°F.

### ***14.2.3 Modular Expansion Joints***

When dividing a bridge into segments will not reduce the joint movement to less than 4 inches, use modular expansion joints. Provide a joint setting schedule with modular joints that lists the opening the joint should have at different construction temperatures. Show joint openings for a temperature range from 45°F to 90°F in 15°F increments.

Note that conventional modular joints are one-directional units. Bridges with skews or horizontal curvature may require the use of "swivel" modular joints. These accommodate lateral movement as well as longitudinal movements.

### ***14.2.4 Expansion Joint Detailing***

Show the elevation at the top of the extrusion at crown break points, gutter lines, and the start and end of curved sections. Dimension the lengths for straight and curved portions of the expansion joint.

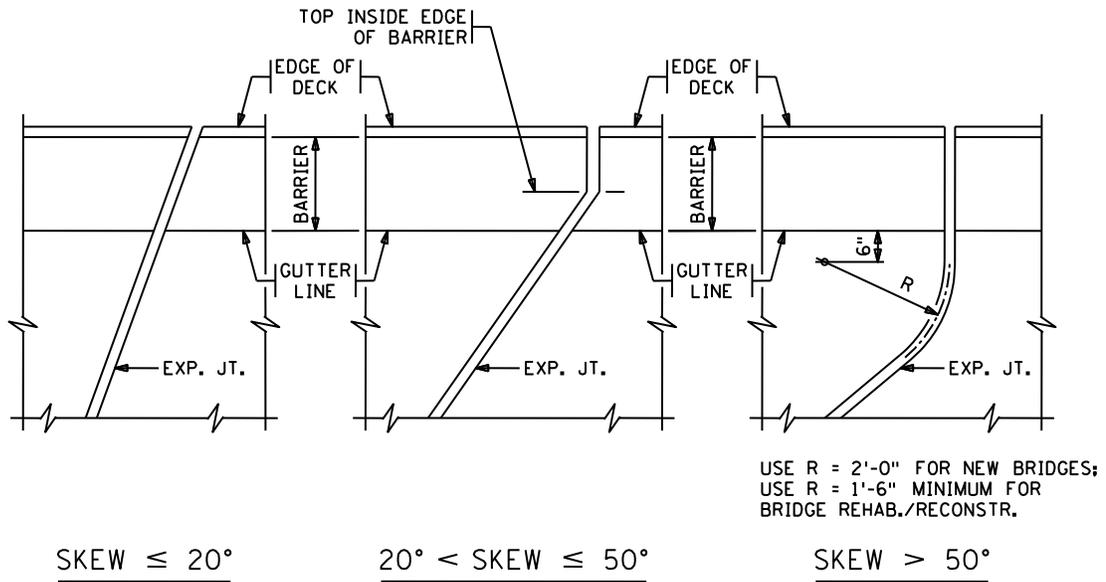
For skews up to 20°, detail expansion joint as straight from edge of deck to edge of deck. See Figure 14.2.4.1.

For skews greater than 20° and up to 50°, detail expansion joint opening as straight between the top inside edge of barriers. Kink the joint opening at top inside edge of barriers so it is normal with outside edge of deck. See Figure 14.2.4.1.

For skews greater than 50°, curve the expansion joint ends. Use a 2'-0" radius for new bridges. A minimum radius of 1'-6" is allowed on bridge rehabilitation/reconstruction projects. Terminate the curved section 6 inches from gutter line. See Figure 14.2.4.1.

Use bend-up details for all bridges with curbs or barriers. For bridges with skewed joints, verify that the bend-up details in the barrier do not project out of the front face of the barrier.

Use snowplow protection for expansion joint devices (Bridge Details Part II Fig. 5-397.628) when joints are skewed greater than 15° and less than 50°.



**Figure 14.2.4.1**  
**Expansion Joint Details**

**14.3 Bearings**

The purpose of a bridge bearing is to transmit loads from the superstructure to the substructure while facilitating translation and rotation. Four types of bearings are typically used:

- 1) Expansion Bearing:
  - Transfers vertical load
  - Allows lateral movement in two directions
  - Allows longitudinal rotation
- 2) Guided Expansion Bearing:
  - Transfers vertical load and lateral load in one direction
  - Allows lateral movement in one direction
  - Allows longitudinal rotation
- 3) Limited Expansion Bearing:
  - Transfers vertical load and lateral load
  - Allows limited lateral movement in one direction
  - Allows longitudinal rotation

- 4) Fixed Bearing:
- Transfers vertical load and lateral load
  - Resists lateral movement
  - Allows longitudinal rotation

All of Minnesota is located in Seismic Zone 1. See Article 3.7 of this manual for Seismic Zone 1 requirements for fixed and expansion bearings.

#### ***14.3.1 Loads and Movements***

Design bearings for movements associated with a temperature range of 150°F (-30°F to 120°F) and a base construction temperature of 45°F.

Design elastomeric bearings for service loads and without Dynamic Load Allowance (IM).

#### ***[14.6.1]***

Uplift at bearings is not permitted. Check bearings for uplift using the Strength I load combination with the minimum load factor for dead load.

For post-tensioned concrete bridges, consider the movement due to concrete creep for design of the bearings and expansions joints.

#### ***14.3.2 Bearing Details***

Identify the type of bearing used at each support location on the superstructure framing plan.

For bearing components, the length is measured parallel to the centerline of the beam and the width is measured perpendicular to the centerline of the beam.

Check the dimensions of the bearing. Check that bearings have adequate clearance to other bearings (pier locations), are consistent with the beam end details (pier and abutment locations), and have adequate clearance to vertical faces of supporting elements. For fixed bearings, provide a minimum of 1 inch clear from the face of the bearing seat to the bearing pad or masonry plate. For expansion bearings, increase this minimum dimension to 3 inches.

Locate bearing anchor rods to permit field drilling of holes and provide 2 inch minimum clearance to reinforcement in bridge seat.

Bearings typically provide a modest amount of lateral restraint. However, designers must consider whether or not additional restraint needs to be provided. Typically, this additional restraint is provided by

reinforced concrete guide lugs in the substructure or slotted hole fixed bearing assemblies adjacent to the center beam at expansion piers and abutments for bridges on large skewes or curves. A 1 inch clear dimension must be provided between elements for either of these restraint methods. Provide additional restraint for pedestrian bridges.

The service life of bearings is less than the anticipated service life of a bridge. To simplify future maintenance operations and potential replacement, provide adequate clearance for the installation of jacks (at least 6 inches) and also provide a jacking load path. The load path may involve properly designed and detailed diaphragms or a suitable superstructure element.

When the slope of steel beam or plate girder superstructures exceeds 3%, incorporate tapered sole plates into the bearings. When the slope of prestressed beam superstructures exceeds 3%, incorporate tapered bearing plates into the bearings, using standard detail B309 as a guide.

For bridge bearings that have a masonry plate (e.g. - new disc bearings or existing lubricated bronze plate bearings being replaced in kind), include a 1/8 inch 60 durometer plain elastomeric pad between the bridge seat and new masonry plate to provide proper load distribution.

### **14.3.3 Elastomeric Bearings**

Use of elastomeric bearings is preferred over other types of bearings. Fixed and expansion elastomeric bearing types are used most frequently.

MnDOT's standard elastomeric bearings include:

- Detail B304: Fixed bearing used on bridge repair projects for replacement of existing bearings where there is insufficient height available for a curved plate bearing assembly, consisting of a steel reinforced elastomeric pad with bearing plate and anchor rods, sized to match the height of the existing bearing.
- Detail B305: Fixed bearing used at integral abutments or at piers with continuity diaphragms consisting of a plain elastomeric pad.
- Detail B309: Fixed bearing used at integral abutments or at piers with continuity diaphragms when grades exceed 3%, consisting of a plain elastomeric pad with a tapered bearing plate.
- Detail B310: Fixed bearing for prestressed concrete beams consisting of a plain elastomeric pad with a curved plate to allow rotation, and anchor rods for fixity.
- Detail B311: Expansion bearing for prestressed concrete beams consisting of a steel reinforced elastomeric pad with a curved plate to allow rotation.

- Detail B354: Fixed bearing for steel beams consisting of a plain elastomeric pad with a curved plate to allow rotation, and anchor rods for fixity.
- Detail B355: Expansion bearing for steel beams consisting of a steel reinforced elastomeric pad with a curved plate to allow rotation.

*Note that the use of plain elastomeric pads is currently limited per Memo to Designers (2012-01) due to issues of excessive pad deformation. For all fixed curved plate bearing assemblies (Details B310 and B354), plain elastomeric bearing pads are replaced with cotton-duck bearing pads of the same size as required for a plain pad. However, the guidance regarding plain elastomeric pads within this article has been retained until a final policy decision is made regarding their use.*

#### **14.3.3.1 Design**

Use the tables found in Article 14.7 of this manual whenever possible for consistency and economy among bearing designs.

Design elastomeric bearings using Method A of the AASHTO LRFD Specifications.

##### **[Table 14.7.6.2-1]**

Design using an elastomer hardness of 60 durometers. The minimum shear modulus (G) for this material is 130 psi. The maximum shear modulus is 200 psi.

Except for special designs, use steel with a yield strength  $F_y$  equal to 36 ksi for all bearing assembly plates.

For MnDOT bridges with curved plate bearings, rotations need not be considered in the design.

For maximum compressive stress checks, use the minimum shear modulus value.

Holes are not permitted in elastomeric bearings.

##### **14.3.3.1.1 Size and Stability**

Although not an AASHTO requirement, MnDOT has historically used the following limits for the shape factor,  $S$ , for plain pads and internal laminates of steel reinforced pads with good success:

$$5.0 \leq S \leq 10.0$$

For fixed bearings, use  $\frac{1}{2}$  inch or  $\frac{3}{4}$  inch thickness plain pads. For expansion bearings, use  $\frac{3}{8}$  inch,  $\frac{1}{2}$  inch, or  $\frac{3}{4}$  inch thickness internal laminates with  $\frac{1}{8}$  inch thick steel reinforcing plates and  $\frac{1}{4}$  inch thick cover layers.

Round dimensions for elastomeric bearings to the nearest 2 inch increment. For "RB", "M", and "MN" series prestressed beams, the minimum length (A) is 12 inches and the minimum width (B) is 24 inches. For "MW" series prestressed beams, the minimum length (A) is 16 inches and the minimum width (B) is 36 inches. For steel beams, the minimum length (A) is 8 inches. The width (B) shall not be less than the bottom flange width and not more than 2 inches greater than the bottom flange width for steel beams.

Based on the past performance of elastomeric bearings, MnDOT places a limit on the plan aspect ratio of a bearing. The length (A) is limited by the following equation:

$$B \leq 2.5 \cdot A$$

**[14.7.6.3.6]**

Additionally, the total elastomer thickness for the bearing ( $h_{rt}$ ) must be no more than  $\frac{1}{3}$  of the bearing pad length and width:

$$h_{rt} \leq \frac{A}{3} \text{ and } \frac{B}{3}$$

**14.3.3.2 Fixed Bearings**

**[14.7.6.3.2]**

Design fixed elastomeric bearings for a maximum compressive stress of 0.880 ksi. This includes a 10% increase for fixity.

Provide transverse fixity for  $\frac{2}{3}$  of beams at fixed piers or fixed abutments for widths along skew greater than 70'-0".

**14.3.3.3 Expansion Bearings**

**[14.7.6.3.2]**

**[14.7.6.3.4]**

Design expansion elastomeric bearings to be steel-reinforced, with a maximum compressive stress equal to the lesser of 1.25GS or 1.25 ksi.

In order to accommodate shear deformation in the pad due to thermal movement, the total height or thickness of elastomer ( $h_{rt}$ ) must be greater than twice the maximum design movement. The LRFD Specifications list a load factor of 1.2 to be used for thermal movement calculations. However, based on past performance of bearings, use a load factor of 1.3 with half the design temperature range (75°F) when computing movement  $\Delta_s$  for the shear deformation check.

**[Table 3.4.1-1]**

Timely delivery of elastomeric bearings has been an issue in the past. In an effort to improve availability and encourage fabricators/contractors to stockpile pads, the number of laminates has been standardized into two groupings for "RB", "M", and "MN" series prestressed beam expansion elastomeric bearings. Where possible:

- For design movements  $\Delta_s \leq 1.00"$ , use a 12" x 24" pad with 3 - 1/2" thick laminates
- For pads where  $1.00" < \text{design movement } \Delta_s \leq 1.75"$ , use a 12" x 24" pad with 6 - 1/2" laminates

#### 14.3.3.3.1

##### *Minimum*

##### *Compressive Load*

##### *[C14.8.3.1]*

LRFD Article C14.8.3.1 states that bearings should be anchored securely to the support to prevent their moving out of place. It further states that elastomeric bearings may be left without anchorage provided adequate friction is available and that a design coefficient of friction equal to 0.20 may be assumed between elastomer and concrete or steel. The minimum horizontal resistance to slippage of the bearing is:

$$H_{bres} = 0.20 \cdot P_{min}$$

The factored horizontal shear force  $H_{bu}$  generated in the bearing due to temperature movement is:

##### *[14.6.3.1]*

$$H_{bu} = G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

Equating  $H_{bres}$  and  $H_{bu}$  and solving for the minimum compressive load,  $P_{min}$  results in:

$$P_{min} \geq 5 \cdot G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

For the minimum compressive load check, use the maximum shear modulus value (0.200 ksi). For calculation of  $\Delta_u$ , LRFD Article 3.4.1 specifies a load factor of 1.2. However, based on past performance of bearings, use a load factor of 1.0 with half the design temperature range (75°F) to calculate  $\Delta_u$ .

If the check is not satisfied, revise the number and/or thickness of the laminates as needed. If the requirement still cannot be met, the standard curved plate expansion bearing assemblies (B311 and B355) contain a 3/8" x 3/8" bar welded to the bearing plate. This can be considered as a mechanism that secures the pads.

**14.3.4 Disc Bearings**

Use disc bearings where the loads are too high or the movements and rotations are too large to be readily accommodated with elastomeric bearings.

To reduce the possibility of generating large lateral forces in wide bridges supported on disc bearings, do not use guided or fixed bearings for beam lines outside of the center 45 feet of the bridge (distance measured along the substructure).

Due to a variety of preferences among disc bearing fabricators, explicit bearing details are not provided in the plans. Instead, provide a schematic of the bearings and all applicable design loads and movements in the plans. Using the provided data, the fabricator will determine the size of all of the bearing components, from the masonry plate to the sole plate.

Contact bearing fabricators to determine estimated bearing assembly heights for inclusion in the bridge plan. Also provide the appropriate standard plan note (see Appendix 2-C.J) in the bridge plan regarding the estimated bearing assembly heights and adjustments to seat elevations.

Fixed disc bearings allow for limited rotation, but no movement.

Guided expansion disc bearings allow for free movement in one direction and provide rotational capacity. However, movement perpendicular to the free movement direction is restrained. For curved bridges, assume the free movement direction to be along a chord connecting the ends of the beam. Guide bars must resist a minimum of 15% of the vertical service limit state load applied to the bearing.

Expansion disc bearings provide for rotation and unguided movement in all horizontal directions.

For computation of movement for design of disc bearings, use a load factor of 1.2.

**14.3.5 Other Types of Bearings**

**Pot Bearings**

Do not use pot bearings on roadway bridges.

**Steel Bearings**

This type of bearing does not contain elastomeric components to accommodate horizontal movement. Rather, horizontal movement takes place at the interface of a machined masonry plate and a lubricated

bronze plate. Bridge Details Part I B351, B352, and B353 detail fixed, expansion, and guided expansion steel bearings respectively. They have all been archived, but can be retrieved if necessary for a repair plan. Note that these bearings are used for bridge repair projects only and are not for new construction.

Modify the standard bearings as necessary to accommodate unusually wide flanges or to provide movement capacities greater than those permitted with the standard details.

Check the clearances on the guide bars for curved bridges.

To reduce the possibility of generating large lateral forces in wide bridges supported on steel bearings, do not use guided or fixed bearings for beam lines outside of the center 45'-0" of the bridge (distance measured along the substructure).

#### **Bearings for Railroad Bridges**

Due to the extremely large loads associated with railroad bridges, spherical bearings, rocker bearings, disc bearings, or pot bearings are normally required. Rocker bearings may be considered for other applications where there is a combination of large load and large movement.

#### **14.4 Curved Plate Design**

##### **Width**

For prestressed concrete beams, set the width (H) equal to the bearing pad width (B) plus 2 inches. The width may change slightly (2 inches to 4 inches) for special designs. For steel beams, set the width equal to the bearing pad width (B).

##### **Thickness**

Use the LRFD design method for determining the curved plate thickness. Although AASHTO LRFD allows the nominal flexural resistance of a section to be taken as the plastic moment of a section, MnDOT limits the nominal flexural resistance to the yield moment. For steel elements in flexure use a resistance factor,  $\phi_f$ , equal to 1.0.

#### **[6.5.4.2]**

The all-around weld, together with the friction between plates, causes the curved plate and bearing plate to act compositely. Therefore, the thickness for design can be considered to include the curved plate thickness plus the bearing plate thickness. The minimum thickness for curved plates is  $1\frac{1}{4}$  inches. When greater thickness is required, increase plate thickness in  $\frac{1}{4}$  inch increments.

**Length**

The minimum length (G) for the curved plate is  $4\frac{1}{2}$  inches. The next permitted length is 6 inches, after which the length may be increased by increments of 2 inches up to a maximum of 12 inches. If, when designing the bearing plate, the required bearing plate thickness exceeds 2 inches, increase the length of the curved plate to reduce the length of the cantilever for the bearing plate design. Increase the curved plate length until the required bearing plate thickness alone and the required plate thickness for the curved plate based on composite design are approximately equal.

**Radius**

The radius of curved plates is to be no less than 16 inches. Check contact stresses to make sure that an adequate radius is provided. Based on past satisfactory performance of curved plate bearing assemblies, use LRFD Equations C14.7.1.4-1 and C14.7.1.4-2 for determination of curved plate radius. If the resulting radius exceeds 24 inches, a special design must be completed using LRFD Equation 14.7.1.4-1 and steel with a yield strength  $F_y$  equal to 50 ksi.

[14.7.1.4]

**14.5 Bearing Plate  
Design**

**Width**

For prestressed concrete beams, set the width (E) equal to the curved plate width (H) plus 1 inch for expansion bearings. For fixed bearings, set the width (E) equal to the beam bottom flange width plus 8 inches. For steel beams, set the width (E) equal to the curved plate width (B) plus 2 inches for expansion bearings and plus 10 inches for fixed bearings.

**Length**

Set the length of the bearing plate (C) 2 inches larger than the bearing pad length (A).

**Thickness**

Use the LRFD design method for determining the bearing plate thickness. Although AASHTO LRFD allows the nominal flexural resistance of a section to be taken as the plastic moment of a section, MnDOT limits the nominal flexural resistance to the yield moment. For steel elements in flexure use a resistance factor,  $\phi_f$ , equal to 1.0.

[6.5.4.2]

The minimum thickness for bearing plates is  $1\frac{1}{2}$  inches. When greater thickness is required, increase plate thickness in  $\frac{1}{4}$  inch increments.

**14.6 Sole Plate  
Design (Steel  
Beams)**

**Width**

Set the width of the sole plate 2 inches larger than the curved plate width (B). The width cannot be equal to the beam flange width because of the fillet weld used to attach the sole plate to the flange. Increase the sole plate width by 1 inch if this occurs.

**Length**

The minimum length is 6 inches. Also, the length shall not be less than the curved plate length (G).

**Thickness**

The minimum sole plate thickness is  $1\frac{1}{4}$  inches. When greater thickness is required, increase plate thickness in  $\frac{1}{8}$  inch increments.

When the bearing pad width exceeds the bottom flange width, the sole plate must be designed as a cantilever to resist the load from the pad that extends outside the flange. Use the LRFD design method. Although AASHTO LRFD allows the nominal flexural resistance of a section to be taken as the plastic moment of a section, MnDOT limits the nominal flexural resistance to the yield moment. For steel elements in flexure use a resistance factor,  $\phi_f$ , equal to 1.0.

[6.5.4.2]

**14.7 Tables**

The following tables contain standard curved plate bearing designs for prestressed concrete and steel beam superstructures based on the guidance given in this manual.

Table 14.7.1	Fixed Curved Plate Bearing Assembly for Prestressed Concrete Beams (B310)
Table 14.7.2	Expansion Curved Plate Bearing Assembly for Prestressed Concrete Beams (B311)
Table 14.7.3	Elastomeric Bearing Pad Thickness for Expansion Curved Plate Bearing Assembly for Prestressed Concrete Beams (B311)
Table 14.7.4	Fixed Curved Plate Bearing Assembly for Steel Beams (B354)
Table 14.7.5	Expansion Curved Plate Bearing Assembly for Steel Beams (B355)
Table 14.7.6	Elastomeric Bearing Pad Thickness for Expansion Curved Plate Bearing Assembly for Steel Beams (B355)

The curved plate thicknesses, the bearing plate thicknesses, and the steel beam sole plate thicknesses given in the tables were designed for the LRFD Strength I limit state by applying the live load factor of 1.75 to the

maximum service load determined for the elastomeric pad design. This ensures a conservative design for the plates without knowing the exact mix of dead load and live load.

Use the tables whenever possible to increase consistency and economy among bearing designs. When calculated loads and/or movements fall outside the limits given in the table, two options are available to designers:

**[14.7.1.4]**

- 1) Complete a special elastomeric bearing design. For this case, use LRFD Equation 14.7.1.4-1 for determination of curved plate radius. Also, use steel with a yield strength equal to 50 ksi for the curved plate. Modify the B-Detail by specifying that the curved plate shall comply with MnDOT Spec. 3310.
- 2) Use a disc bearing.

**Table 14.7.1**  
**Fixed Curved Plate Bearing Assembly for "RB", "M", and "MN"**  
**Series Prestressed Concrete Beams (B310) ①**

Max Service DL+LL (kips)	Bearing Pad Size (in)		Plain Pad Thickness (in)	Shape Factor	Bearing Plate Size (in) ③			Curved Plate Size (in) ③			Minimum Radius (in)
	A	B			C	E	F	G	H	J	
253	12	24	1/2	8.0	14	②	1 1/2	4 1/2	26	1 1/4	16
295	14	↓	↓	8.8	16	↓	↓	6	↓	↓	↓
337	16	↓	↓	9.6	18	↓	2	↓	↓	↓	↓
380	18	↓	3/4	6.9	20	↓	↓	8	↓	↓	↓
422	20	↓	↓	7.3	22	↓	2 1/4	↓	↓	↓	20

- ① Table does not apply to "MW" series beams. A special design is required.
- ② 34" for all "RB" series beams.  
34" for all "M" series I-beams.  
38" for all "MN" series I-beams.
- ③ Plates are conservatively designed for  $1.75 \cdot (\text{Max Service DL} + \text{LL})$ .

**Table 14.7.2**  
**Expansion Curved Plate Bearing Assembly for "RB", "M", and "MN"**  
**Series Prestressed Concrete Beams (B311) ①**

Max Service DL+LL (Kips)	Bearing Pad Size (in)		Laminate Thickness (in)	Max. Number of Laminates ②	Shape Factor	Bearing Plate Size (in) ③			Curved Plate Size (in) ③			Minimum Radius (in)
	A	B				C	E	F	G	H	J	
300	12	24	1/2	7	8.0	14	27	1 1/2	4 1/2	26	1 1/4	16
360	12	↓	↓	7	8.0	14	↓	1 3/4	↓	↓	↓	↓
420	14	↓	↓	8	8.8	16	↓	↓	6	↓	↓	19

- ① Table does not apply to "MW" series beams. A special design is required.
- ② See Table 14.7.3 for determination of required number of laminates.
- ③ Plates are conservatively designed for  $1.75 \cdot (\text{Max Service DL} + \text{LL})$ .

**Table 14.7.3**  
**Elastomeric Bearing Pad Thickness for Expansion Curved Plate**  
**Bearing Assembly for Prestressed Concrete Beams (B311) ①②**

Interior Laminate Thickness (in)	D (in) ③	Number of Laminates	Total Elastomer Thickness, $h_{rt}$ (in) ③	Maximum Movement $\Delta_s$ (in) ④
1/2"	2 1/2	3 ⑤	2	1
	4 3/8	6 ⑤	3 1/2	1 3/4
	5	7	4	2
	5 5/8	8	4 1/2	2 1/4

- ① Table does not apply to "MW" series beams. A special design is required. Table is based on requirements of AASHTO LRFD Bridge Design Specifications Article 14.7.6.3.4:  $h_{rt} \geq 2\Delta_s$ . Engineer must also check that the minimum compressive load requirement (discussed in Article 14.3.3.3.1) is satisfied. Specifically:

$$P_{min} \geq 5 \cdot G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

where  $P_{min}$  is the minimum factored load ( $0.9 \cdot DC + 1.75 \cdot LL_{min}$ ),  $G$  is equal to the maximum shear modulus value (0.200 ksi),  $A_{pad}$  is the plan area of the bearing pad, and  $\Delta_u$  is the movement of the bearing pad from the undeformed state using a 75°F temperature

- ② Engineer must also check the elastomeric bearing pad for compression deflection based on the requirements from AASHTO LRFD Bridge Design Specifications Articles 14.7.6.3.3 and 14.7.5.3.6.
- ③ Pad thickness  $D$  includes  $h_{rt}$  and 1/8" steel reinforcement plates. Total elastomer thickness  $h_{rt}$  includes interior laminates plus 1/4" cover layers.
- ④ Maximum movement  $\Delta_s$  is the movement of the bearing pad from the undeformed state to the point of maximum deformation. Use a 75°F temperature change with a 1.3 load factor for calculation of maximum movement.
- ⑤ For "RB", "M", and "MN" series prestressed beam expansion elastomeric bearings, the number of laminates has been standardized for the movements that are most often encountered.
- If  $\Delta_s \leq 1.00"$ , use 3 - 1/2" laminates.
  - If  $1.00" < \Delta_s \leq 1.75"$ , use 6 - 1/2" laminates.

Table 14.7.4 – Fixed Curved Plate Bearing Assembly for Steel Beams (B354)

Beam Flange Min. Width (in)	Beam Flange Max. Width (in)	Max. Service DL+LL (kips)	Bearing Pad Size (in)		Plain Pad Thick. (in)	Shape Factor	Bearing Plate Size (in) ①				Curved Plate Size (in) ①			Min. Radius (in)	Sole Plate Size (in)		
			A	B			C	E	F	G	B	H	Length		Width	Thick.	
12	14	81	8	14	1/2	5.1	10	24	1 1/2	4 1/2	14	1 1/4	16	6	16	1 1/4	
↓	↓	116	10	↓	↓	5.8	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	147	12	↓	↓	6.5	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	172	14	↓	↓	7.0	16	↓	1 3/4	↓	↓	↓	↓	↓	↓	↓	
14	16	97	8	16	1/2	5.3	10	26	1 1/2	4 1/2	16	1 1/4	16	6	18	1 1/4	
↓	↓	140	10	↓	↓	6.2	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	168	12	↓	↓	6.9	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	197	14	↓	↓	7.5	16	↓	1 3/4	↓	↓	↓	↓	↓	↓	↓	
↓	↓	225	16	↓	↓	8.0	18	↓	↓	6	↓	↓	17	↓	↓	↓	
16	18	113	8	18	1/2	5.5	10	28	1 1/2	4 1/2	18	1 1/4	16	6	20	1 1/4	
↓	↓	158	10	↓	↓	6.4	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	190	12	↓	↓	7.2	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	221	14	↓	↓	7.9	16	↓	1 3/4	↓	↓	↓	↓	↓	↓	↓	
↓	↓	253	16	↓	↓	8.5	18	↓	2	6	↓	↓	↓	↓	↓	↓	
↓	↓	285	18	↓	↓	9.0	20	↓	↓	8	↓	↓	19	8	↓	↓	
18	20	130	8	20	1/2	5.7	10	30	1 1/2	4 1/2	20	1 1/4	16	6	22	1 1/4	
↓	↓	176	10	↓	↓	6.7	12	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	211	12	↓	↓	7.5	14	↓	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	246	14	↓	↓	8.2	16	↓	1 3/4	↓	↓	↓	↓	↓	↓	↓	
↓	↓	281	16	↓	↓	8.9	18	↓	2	6	↓	↓	↓	↓	↓	↓	
↓	↓	316	18	↓	↓	9.5	20	↓	↓	8	↓	↓	18	8	↓	↓	
↓	↓	352	20	↓	↓	10.0	22	↓	2 1/4	↓	↓	↓	22	↓	↓	↓	

**Table 14.7.4 (Cont.) – Fixed Curved Plate Bearing Assembly for Steel Beams (B354)**

Beam Flange Min. Width (in)	Beam Flange Max. Width (in)	Max. Service DL+LL (kips)	Bearing Pad Size (in)		Plain Pad Thick. (in)	Shape Factor	Bearing Plate Size (in) ①				Curved Plate Size (in) ①			Min. Radius (in)	Sole Plate Size (in)		
			A	B			C	E	F	G	B	H	Length		Width	Thick.	
20	22	193	10	22	1/2	6.9	12	32	1 1/2	4 1/2	22	1 1/4	16	6	24	1 1/4	
→	→	232	12	→	→	7.8	14	→	→	→	→	→	→	→	→	→	
→	→	271	14	→	→	8.6	16	→	1 3/4	→	→	→	→	→	→	→	
→	→	309	16	→	→	9.3	18	→	2	6	→	→	→	→	→	→	
→	→	348	18	→	→	9.9	20	→	→	8	→	→	17	8	→	→	
→	→	387	20	→	3/4	7.0	22	→	2 1/4	→	→	→	21	→	→	→	
→	→	426	22	→	→	7.3	24	→	2 1/2	→	→	→	25	→	→	→	
22	24	211	10	24	1/2	7.1	12	34	1 1/2	4 1/2	24	1 1/4	16	6	26	1 1/4	
→	→	253	12	→	→	8.0	14	→	→	→	→	→	→	→	→	→	
→	→	295	14	→	→	8.8	16	→	1 3/4	→	→	→	→	→	→	→	
→	→	337	16	→	→	9.6	18	→	2	6	→	→	→	→	→	→	
→	→	380	18	→	3/4	6.9	20	→	→	8	→	→	→	8	→	→	
→	→	422	20	→	→	7.3	22	→	2 1/4	→	→	→	20	→	→	→	
→	→	464	22	→	→	7.7	24	→	2 1/2	→	→	→	24	→	→	→	
24	26	274	12	26	1/2	8.2	14	36	1 1/2	4 1/2	26	1 1/4	16	6	28	1 1/4	
→	→	320	14	→	→	9.1	16	→	1 3/4	→	→	→	→	→	→	→	
→	→	366	16	→	→	9.9	18	→	2	6	→	→	→	→	→	→	
→	→	411	18	→	3/4	7.1	20	→	→	8	→	→	→	8	→	→	
→	→	457	20	→	→	7.5	22	→	2 1/4	→	→	→	19	→	→	→	
→	→	503	22	→	→	7.9	24	→	2 1/2	→	→	→	23	→	→	→	

① Plates are conservatively designed for 1.75 · Max Service DL+ LL

**Table 14.7.5 – Expansion Curved Plate Bearing Assembly for Steel Beams (B355)**

Beam Flange Min. Width (in)	Beam Flange Max. Width (in)	Max. Service DL+LL (kips)	Bearing Pad Size (in)		Laminate Thick. (in)	Max. Number of Laminates ①	Shape Factor	Bearing Plate Size (in) ②			Curved Plate Size (in) ②			Min. Radius (in)	Sole Plate Size (in)		
			A	B				C	E	F	G	B	H		Length	Width	Thick.
12	14	123	8	14	3/8	5	6.8	10	16	1 1/2	4 1/2	14	1 1/4	6	16	1 1/4	
↓	↓	175	10	↓	↓	7	7.8	12	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	210	12	↓	↓	9	8.6	14	↓	1 3/4	↓	↓	↓	↓	↓	↓	
14	16	147	8	16	3/8	5	7.1	10	18	1 1/2	4 1/2	16	1 1/4	6	18	1 1/4	
↓	↓	200	10	↓	↓	7	8.2	12	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	240	12	↓	↓	9	9.1	14	↓	1 3/4	↓	↓	↓	↓	↓	↓	
16	18	172	8	18	3/8	5	7.4	10	20	1 1/2	4 1/2	18	1 1/4	6	20	1 1/4	
↓	↓	225	10	↓	↓	7	8.6	12	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	270	12	↓	↓	9	9.6	14	↓	1 3/4	↓	↓	↓	↓	↓	↓	
↓	↓	315	14	↓	1/2	8	7.9	16	↓	↓	6	↓	↓	↓	↓	↓	
18	20	198	8	20	3/8	5	7.6	10	22	1 1/2	4 1/2	20	1 1/4	6	22	1 1/4	
↓	↓	250	10	↓	↓	7	8.9	12	↓	↓	↓	↓	↓	↓	↓	↓	
↓	↓	300	12	↓	↓	9	10.0	14	↓	1 3/4	↓	↓	↓	↓	↓	↓	
↓	↓	350	14	↓	1/2	8	8.2	16	↓	↓	6	↓	↓	↓	↓	↓	
20	22	275	10	22	3/8	7	9.2	12	24	1 1/2	4 1/2	22	1 1/4	6	24	1 1/4	
↓	↓	330	12	↓	1/2	↓	7.8	14	↓	1 3/4	↓	↓	↓	↓	↓	↓	
↓	↓	385	14	↓	↓	8	8.6	16	↓	↓	6	↓	↓	↓	↓	↓	
22	24	300	10	24	3/8	7	9.4	12	26	1 1/2	4 1/2	24	1 1/4	6	26	1 1/4	
↓	↓	360	12	↓	1/2	↓	8.0	14	↓	1 3/4	↓	↓	↓	↓	↓	↓	
↓	↓	420	14	↓	↓	8	8.8	16	↓	↓	6	↓	↓	↓	↓	↓	
24	26	390	12	26	1/2	7	8.2	14	28	1 3/4	4 1/2	26	1 1/4	6	28	1 1/4	
↓	↓	455	14	↓	↓	8	9.1	16	↓	↓	6	↓	↓	↓	↓	↓	

① See Table 14.7.6 for determination of required number of laminates.

② Plates are conservatively designed for 1.75 · Max Service DL+ LL

**Table 14.7.6**  
**Elastomeric Bearing Pad Thickness for Expansion Curved Plate**  
**Bearing Assembly for Steel Beams (B355) ①②**

Interior Laminate Thickness (in)	D (in) ③	Number of Laminates ④	Total Elastomer Thickness, $h_{rt}$ (in) ③	Maximum Movement (in) ⑤
3/8"	1 1/8	1	7/8	7/16
	1 5/8	2	1 1/4	5/8
	2 1/8	3	1 5/8	13/16
	2 5/8	4	2	1
	3 1/8	5	2 3/8	1 3/16
	3 5/8	6	2 3/4	1 3/8
	4 1/8	7	3 1/8	1 9/16
	4 5/8	8	3 1/2	1 3/4
	5 1/8	9	3 7/8	1 15/16
	5 5/8	10	4 1/4	2 1/8
	6 1/8	11	4 5/8	2 5/16
1/2"	1 1/4	1	1	1/2
	1 7/8	2	1 1/2	3/4
	2 1/2	3	2	1
	3 1/8	4	2 1/2	1 1/4
	3 3/4	5	3	1 1/2
	4 3/8	6	3 1/2	1 3/4
	5	7	4	2
	5 5/8	8	4 1/2	2 1/4
	6 1/4	9	5	2 1/2
	6 7/8	10	5 1/2	2 3/4
	7 1/2	11	6	3

① Table is based on requirements of AASHTO LRFD Article 14.7.6.3.4:  $h_{rt} \geq 2\Delta_s$ . Engineer must also check that the minimum compressive load requirement (discussed in Article 14.3.3.3.1 of this manual) is satisfied:

$$P_{min} \geq 5 \cdot G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

where  $P_{min}$  is the minimum factored load ( $0.9 \cdot DC + 1.75 \cdot LL_{min}$ ),  $G$  is equal to the maximum shear modulus value (0.200 ksi),  $A_{pad}$  is the plan area of the bearing pad, and  $\Delta_u$  is the movement of the bearing pad from the undeformed state using a 75°F temperature change with a 1.0 load factor.

- ② Engineer must also check the elastomeric bearing pad for compression deflection based on the requirements from AASHTO LRFD Bridge Design Specifications Articles 14.7.6.3.3 and 14.7.5.3.6.
- ③ Pad thickness  $D$  includes  $h_{rt}$  and 1/8" steel reinforcement plates. Total elastomer thickness  $h_{rt}$  Includes interior laminates plus 1/4" cover layers.
- ④ Engineer must also check that  $S_r^2/n < 22$  to meet requirements of AASHTO LRFD Article 14.7.6.3.4.
- ⑤ Maximum movement  $\Delta_s$  is the movement of the bearing pad from the undeformed state to the point of maximum deformation. Use a 75°F temperature change with a 1.3 load factor for calculation of maximum movement.

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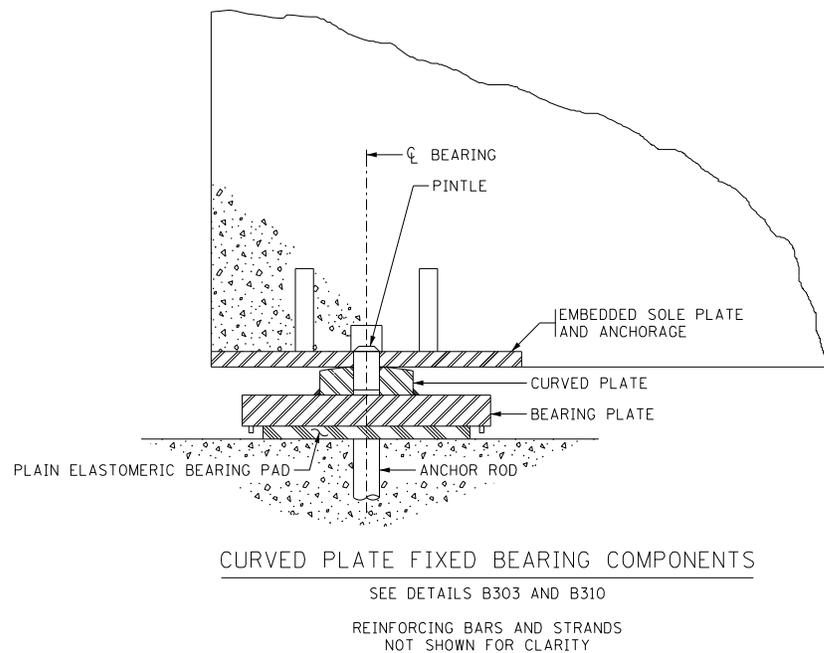
***14.8 Design  
Examples***

Two design examples follow. The first is a fixed elastomeric bearing. The second is an expansion elastomeric bearing.

**14.8.1 Fixed  
Elastomeric  
Bearing Design  
Example  
[14.7.6]**

Note that the use of plain elastomeric pads is currently limited per Memo to Designers (2012-01) due to issues of excessive pad deformation. For all fixed curved plate bearing assemblies (Details B310 and B354), plain elastomeric bearing pads are replaced with cotton-duck bearing pads of the same size as required for a plain pad. However, the following design example has been retained until a final policy decision is made regarding their use.

This example illustrates the design of a fixed curved plate elastomeric bearing for a prestressed concrete beam bridge. The bearing is based on Bridge Details Part I B310. The elastomeric bearing pad is designed using Method A (LRFD Article 14.7.6). Figure 14.8.1.1 shows the bearing components.



**Figure 14.8.1.1**

After the maximum reaction is calculated, the bearing design should be selected from the standard tables found in BDM Article 14.7. If a standard design will not work due to unusual loads or geometric constraints, a custom design will be required.

This example will outline the procedure to custom design a fixed elastomeric bearing. First, design the elastomeric pad. Next determine the steel plate requirements for the rest of the bearing assembly.

**A. Design  
Elastomeric  
Bearing Pad  
[14.7.6]**

The prestressed beam for this example is an MN63, which has a bottom flange width equal to 30 inches. The design loads are given as follows:

$$\text{Dead Load} = P_{dl} = 156 \text{ kips}$$

$$\text{Maximum Live Load} = 97.6 \text{ kips} \quad (\text{Does not include IM})$$

$$\text{Minimum Live Load} = P_{llmin} = 0 \text{ kips}$$

Combining the loads results in the following:

$$\begin{aligned} \text{Maximum service limit state load } P_s &= P_{dl} + P_{llmax} \\ &= 156 + 97.6 = 253.6 \text{ kips} \end{aligned}$$

$$\begin{aligned} \text{Minimum strength limit state load } P_{umin} &= 0.9 \cdot P_{dl} + 1.75 \cdot P_{llmin} \\ &= 0.9 \cdot 156 + 1.75 \cdot 0 \\ &= 140.4 \text{ kips} \end{aligned}$$

Therefore, there is no uplift.

**[Table 14.7.6.2-1]**

MnDOT Spec. 3741 specifies an elastomeric pad with a hardness of 60 durometers. Per LRFD Table 14.7.6.2-1, the shear modulus  $G$  for design ranges from 0.130 to 0.200 ksi.

The minimum bearing pad dimensions for a prestressed beam are:

$$\text{Length } A = 12 \text{ in}$$

$$\text{Width } B = 24 \text{ in}$$

Assuming a plain pad thickness  $D = 0.50$  in,

**[14.7.5.1]**

$$\text{Shape factor } S = \frac{A \cdot B}{2 \cdot D \cdot (A + B)} = \frac{12 \cdot 24}{2 \cdot 0.5 \cdot (12 + 24)} = 8.0$$

**[14.7.6.3.2]**

The allowable compressive stress  $\sigma_{sall}$  for plain pads is the smaller of:

$$\begin{aligned} \sigma_{sall} &= 1.00 \cdot G \cdot S \\ &= 1.00 \cdot 0.130 \cdot 8.0 \\ &= 1.04 \text{ ksi} \end{aligned}$$

$$\text{or } \sigma_{sall} = 0.80 \text{ ksi.} \quad < \text{GOVERNS}$$

The allowable is increased by 10% for a fixed bearing because shear deformation is prevented.

$$\sigma_{sallfixed} = 1.10 \cdot 0.80 = 0.88 \text{ ksi}$$

Then the maximum service limit state stress is:

$$\text{Actual } \sigma_s = \frac{P_s}{A \cdot B} = \frac{253.6}{12 \cdot 24} = 0.88 \text{ ksi} = 0.88 \text{ ksi} \quad \underline{\text{OK}}$$

There are two geometric checks on the bearing pad to ensure that it has good proportions. First, in plan, the length of the long side can be no more the 2.5 times the length of the short side. Second, the height of the elastomeric portion can be no more than  $1/3$  the length of the short side of the pad.

$$2.5 \cdot A = 2.5 \cdot 12 = 30 \text{ in} \geq 24 \text{ in} \quad \underline{\text{OK}}$$

[14.7.6.3.6]

$$\frac{A}{3} = \frac{12}{3} = 4 \text{ in} > 0.50 \text{ in} = h_{rt} \quad \underline{\text{OK}}$$

Therefore, use a 12" x 24" x 1/2" plain pad.

### ***B. Curved Plate Design***

Set the curved plate width 2 inches wider than the bearing pad.

$$H = B + 2 = 24 + 2 = 26 \text{ in}$$

The all-around weld, together with the friction between plates, causes the curved plate and bearing plate to act compositely. Therefore, the thickness for design can be considered to include the curved plate thickness plus the bearing plate thickness.

Begin by checking the thickness for a curved composite plate with a length of 4.5 inches. If, when designing the bearing plate, the required bearing plate thickness exceeds 2 inches, increase the length of the curved plate to reduce the length of the cantilever for the bearing plate design. Increase the curved plate length until the required bearing plate thickness alone and the required plate thickness for the curved plate based on composite design are approximately equal.

$$\text{Curved Plate Length} = G = 4.5 \text{ in}$$

The radius of the contact surface is the first parameter to determine for the curved plate. The radius of the curved plate is a function of the yield strength of the steel and the load intensity.

The sole plate width minus the chamfers at each side is greater than the length of the curved plate. Then the contact length of the sole plate with the curved plate is equal to the length of the curved plate minus the pintles and the associated bevels around each of the pintles. See Figure 14.8.1.2.

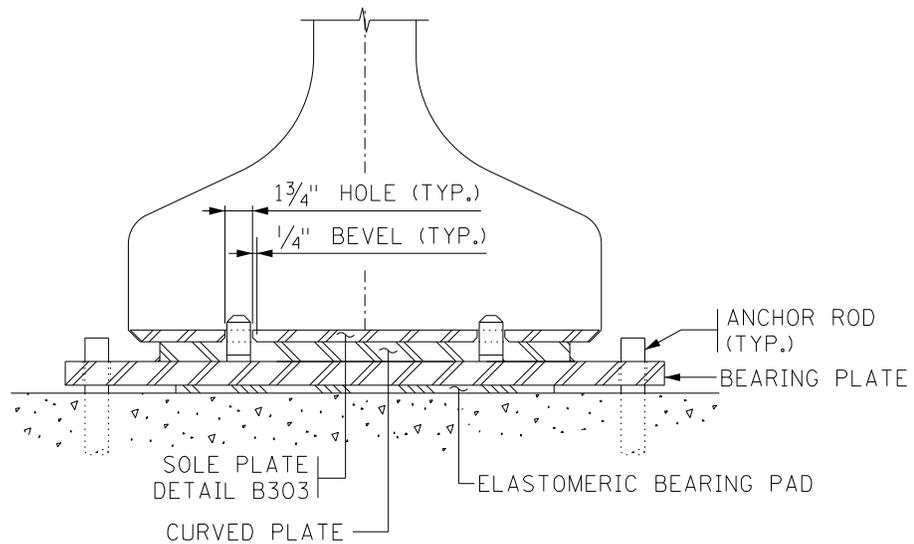


Figure 14.8.1.2

Contact length  $L_{sp}$  is equal to

$$L_{sp} = 26 - 2 \cdot (2.25) = 21.50 \text{ in}$$

[14.7.1.4]

Based on past satisfactory performance of curved plate bearing assemblies, the minimum radius permitted is determined with LRFD Equation C14.7.1.4-1 and C14.7.1.4-2. Start by assuming the diameter  $d$  is 25 inches or less, so use the first equation. Rearranging the equation to solve for diameter results in the following:

$$d_{min} = \frac{20 \cdot p}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left( \frac{P_s}{L_{sp}} \right)}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left( \frac{253.6}{21.50} \right)}{0.6 \cdot (36 - 13)}$$

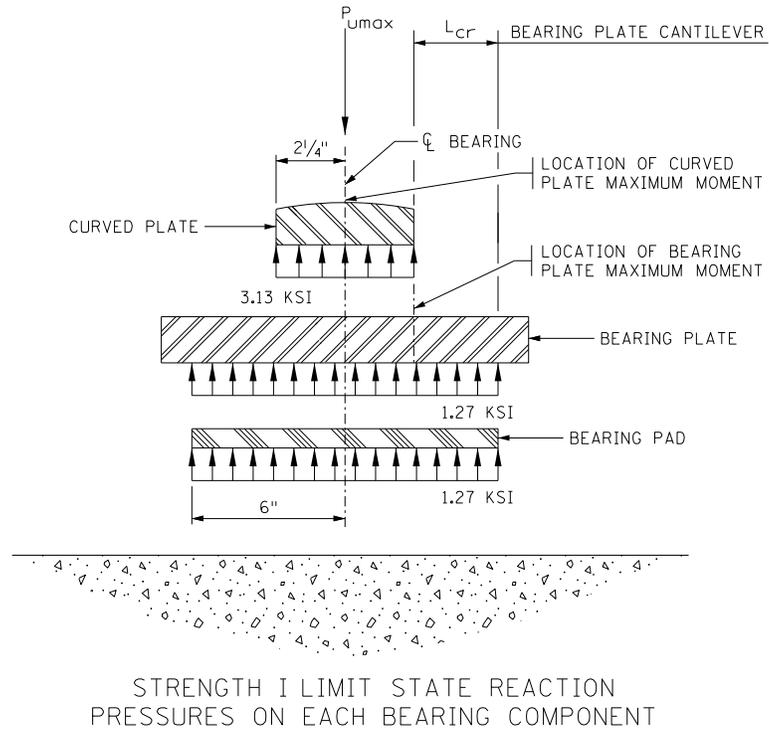
$$= 17.1 \text{ in} < 25.0 \text{ in}$$

The assumption was correct. Then the radius  $R_{min} = 8.55$  inches.

The radius of curved plates is to be no less than 16 inches. Therefore, specify the minimum radius for the curved plate to be 16 inches.

The required thickness of the curved composite plate is based on a simple model in which a uniform pressure is applied to the bottom of the

plate and the reaction is a line load. See Figure 14.8.1.3. Use strength limit state loads for flexural design of the steel plates.



**Figure 14.8.1.3**

$$\begin{aligned}
 \text{Maximum strength limit state load } P_{UMAX} &= 1.25 \cdot P_{DL} + 1.75 \cdot P_{LLMAX} \\
 &= 1.25 \cdot 156 + 1.75 \cdot 97.6 \\
 &= 365.8 \text{ kips}
 \end{aligned}$$

Pressure on the composite plate is:

$$\sigma_{CP} = \frac{P_{UMAX}}{G \cdot H} = \frac{365.8}{4.5 \cdot 26} = 3.13 \text{ ksi}$$

Maximum moment on the composite plate is:

$$M_{UCP} = \sigma_{CP} \cdot \frac{G}{2} \cdot \frac{G}{4} \cdot H = 3.13 \cdot \frac{4.5}{2} \cdot \frac{4.5}{4} \cdot 26 = 206.0 \text{ kip-in}$$

**[6.12.2.2.7]**

Consider the plate to be fully laterally supported. The AASHTO LRFD specifications allow the nominal flexural resistance of a rectangular section to be taken as the plastic moment. However, MnDOT limits the

nominal resistance to the yield moment. Find the required composite plate thickness such that the yield moment,  $M_y$ , of the section will have adequate capacity to resist the design moment,  $M_{ucp}$ .

**[6.5.4.2]**

For steel elements in flexure,  $\phi_f = 1.0$ .

The flexural resistance,  $M_r$ , of the composite plate section is:

$$M_r = \phi_f M_{ncp} = \phi_f M_y = \phi_f S_{cp} F_y$$

The section modulus of the composite plate is:

$$S_{cp} = \frac{H \cdot J^2}{6}$$

where  $J$  = thickness of composite plate

Then by substitution:

$$M_r = \phi_f \cdot \left( \frac{H \cdot J^2}{6} \right) \cdot F_y$$

Set the flexural resistance of the composite plate section equal to the design moment:

$$M_{ucp} = \phi_f \cdot \left( \frac{H \cdot J^2}{6} \right) \cdot F_y$$

Solve for composite plate thickness:

$$J \geq \sqrt{\frac{6 \cdot M_{ucp}}{\phi_f F_y \cdot H}} = \sqrt{\frac{6 \cdot 206 \cdot 0}{1.0 \cdot 36 \cdot 26}} = 1.15 \text{ in}$$

The standard curved plate thickness is 1¼ inches, so composite action does not need to be considered. Use a 1¼ thick curved plate.

**C. Bearing Plate Design**

Per Detail B310, the length (C) is set at 2 inches longer than the pad length. This provides room for the keeper bar to be attached to the bottom of the bearing plate. The width (E) is set 8 inches greater than the beam bottom flange width. This provides room on each side for the anchor rods.

$$E = b_f + 8 = 30 + 8 = 38 \text{ in}$$

$$C = A + 2 = 12 + 2 = 14 \text{ in}$$

The bearing plate is assumed to act as a cantilever (See Figure 14.8.1.3) that carries the maximum strength limit state load to the curved plate. The cantilever length is half the difference in length between the bearing pad and the curved plate.

$$\sigma_{bp} = \frac{P_{umax}}{A \cdot B} = \frac{365.8}{12 \cdot 24} = 1.27 \text{ ksi}$$

$$L_{cr} = \frac{A}{2} - \frac{G}{2} = \frac{12}{2} - \frac{4.5}{2} = 3.75 \text{ in}$$

$$M_{ubp} = \sigma_{bp} \cdot \frac{L_{cr}^2}{2} \cdot E = 1.27 \cdot \frac{3.75^2}{2} \cdot 38 = 339.3 \text{ kip-in}$$

**[6.12.2.2.7]**

Again, the AASHTO LRFD Specifications allow the nominal flexural capacity to be set equal to the plastic moment of the plate. However, MnDOT limits the nominal capacity of the plate to the yield moment. Find the required bearing plate thickness such that the yield moment,  $M_y$ , of the section will have adequate capacity to resist the design moment,  $M_{ubp}$ .

The flexural resistance,  $M_r$ , of the bearing plate section is:

$$M_r = \phi_f M_{nbp} = \phi_f M_y = \phi_f S_{bp} F_y$$

The section modulus of the bearing plate is:

$$S_{bp} = \frac{E \cdot F^2}{6}$$

where  $F$  = thickness of bearing plate

Then by substitution:

$$M_r = \phi_f \cdot \left( \frac{E \cdot F^2}{6} \right) \cdot F_y$$

Set the flexural resistance of the bearing plate section equal to the design moment:

$$M_{ubp} = \phi_f \cdot \left( \frac{E \cdot F^2}{6} \right) \cdot F_y$$

Solve for bearing plate thickness:

$$\text{Min. } F = \sqrt{\frac{6 \cdot M_{ubp}}{\phi_f F_y \cdot E}} = \sqrt{\frac{6 \cdot 339.3}{1.0 \cdot 36 \cdot 38}} = 1.22 \text{ in}$$

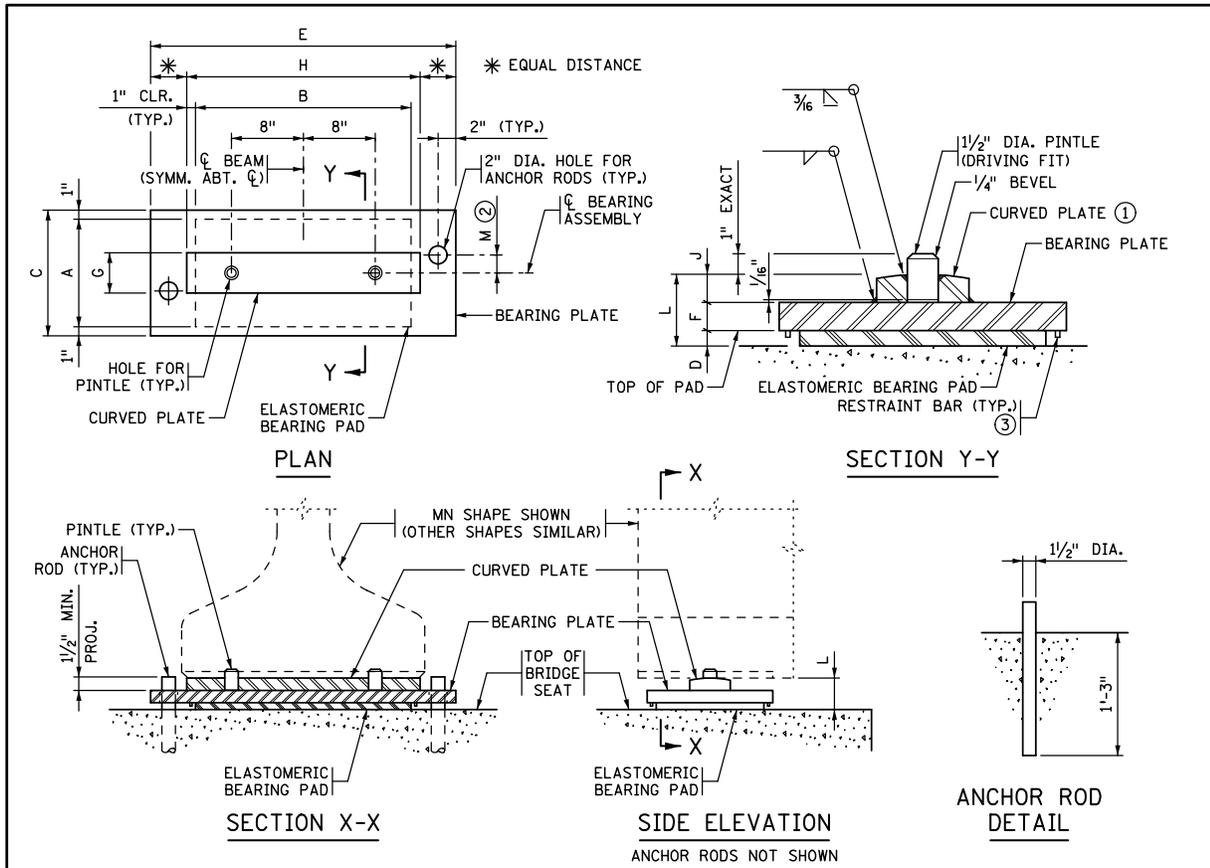
The standard bearing plate thickness is 1½ inches, so use a 1½ thick bearing plate. ||

***D. Anchor  
Rods/Pintles***

The Detail B310 standard set of two 1½ inch anchor rods has a factored load capacity of 72.4 kips and the set of two pintles has a factored load capacity of 100.6 kips. For many projects, such as the superstructure assumed for this design example, the capacity of the anchor rods and pintles will be adequate by inspection. For projects where two or more piers are fixed or where significant longitudinal forces are anticipated, evaluate the capacity of the anchor rods and pintles. ||

The anchor rod offset dimension (M) is to be calculated such that the anchor rods are located along the beam centerline of bearing. In this case, the skew is zero, so  $M = 0$  inches. |

The bearing design is summarized in Figures 14.8.1.4 and 14.8.1.5. ||



ASSEMBLY TYPE	LOCATION	BEAM SIZE	BEARING PAD SIZE			SHAPE FACTOR	BEARING PLATE SIZE			CURVED PLATE SIZE				ANCHOR ROD OFFSET	ASSY. HEIGHT	RESTRAINT PATTERN ③	
			A	B	D		C	E	F	G	H	J	R ①				
F1	PIER	MN63	12"	24"	1/2"	8.0	14"	38"	1 1/2"	4 1/2"	26"	1 1/4"	16"	+/- ②	0"	3 3/4"	A-1

**NOTES:**

- PROVIDE ELASTOMERIC MATERIALS AND PAD CONSTRUCTION PER SPEC. 3741.
- PROVIDE STEEL PLATES PER SPEC. 3306.
- PROVIDE ANCHOR RODS PER SPEC. 3306. GALVANIZE PER SPEC. 3394.
- PROVIDE PINTLES PER SPEC. 3309.

③ REFER TO BEARING PAD RESTRAINT SHEET FOR ADDITIONAL INFORMATION AND DETAILS.

GALVANIZE STRUCTURAL STEEL BEARING ASSEMBLY AFTER FABRICATION PER SPEC. 3394, EXCEPT AS NOTED.

PAYMENT FOR BEARING ASSEMBLY INCLUDES ALL MATERIAL ON THIS DETAIL.

- ① THE MIN. RADIUS IS 16" UNLESS OTHERWISE SPECIFIED IN THE TABLE. THE MAX. RADIUS IS 24". FINISH TO 250 MICRO. THE FINISHED THICKNESS OF THE PLATE MAY BE 1/16" LESS THAN SHOWN.
- ② "+" DENOTES OFFSET AS SHOWN. "-" DENOTES OFFSET OPPOSITE OF SHOWN.

**DESIGN DATA:**  
 MAX. FACTORED SHEAR RESISTANCE:  
 - 50.3 KIPS PER 1/2" DIA. PINTLE  
 - 36.2 KIPS PER 1/2" DIA. ANCHOR ROD

APPROVED: SEPTEMBER 22, 2011	STATE OF MINNESOTA DEPARTMENT OF TRANSPORTATION	REVISED 11-06-2013 11-03-2015 01-05-2017 11-02-2017	DETAIL NO.
<i>Nancy S. Ambberger</i> STATE BRIDGE ENGINEER	<b>CURVED PLATE BEARING ASSEMBLY</b> (PRESTRESSED CONCRETE BEAMS) (FIXED)		<b>B310</b>

Figure 14.8.1.4

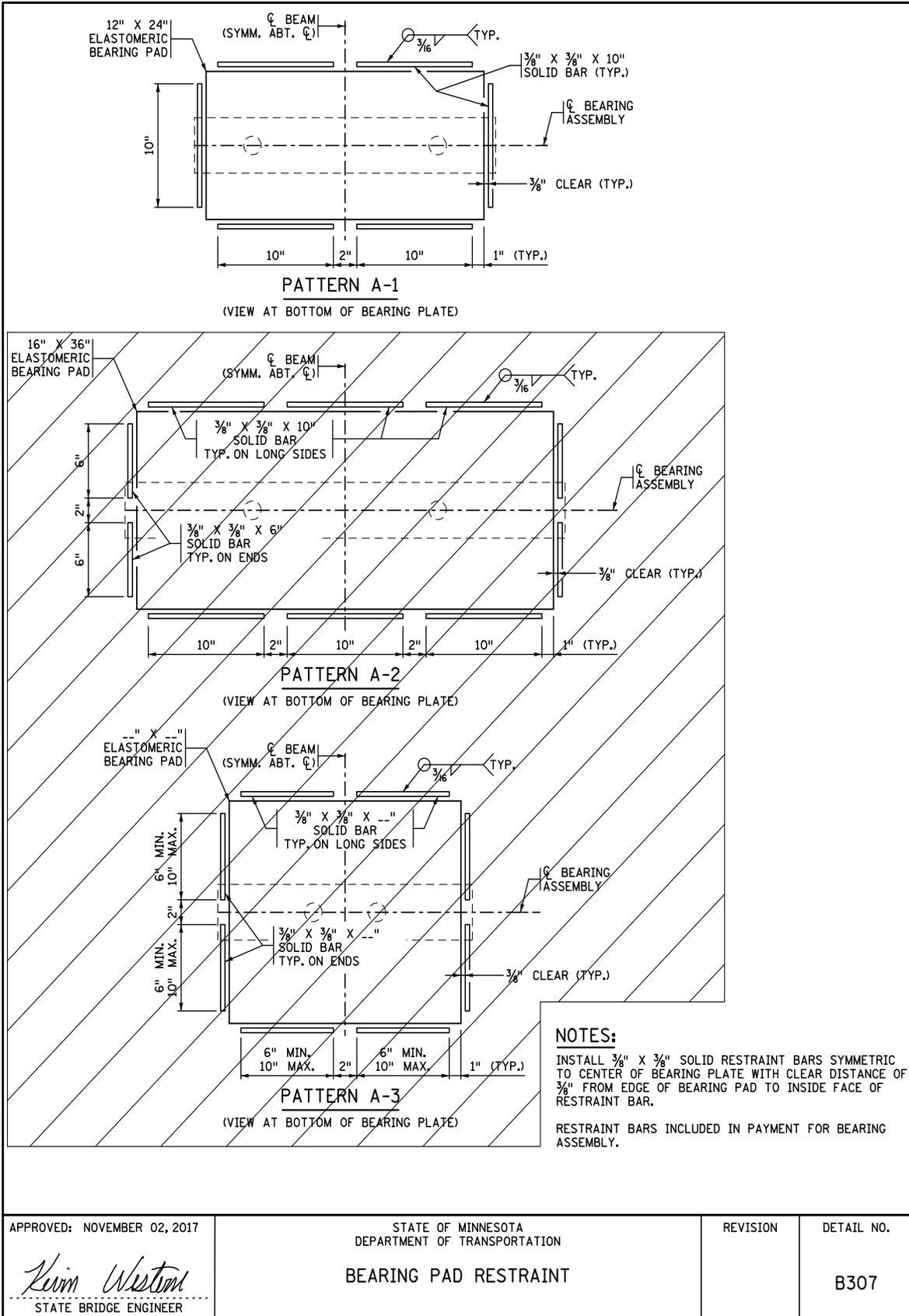
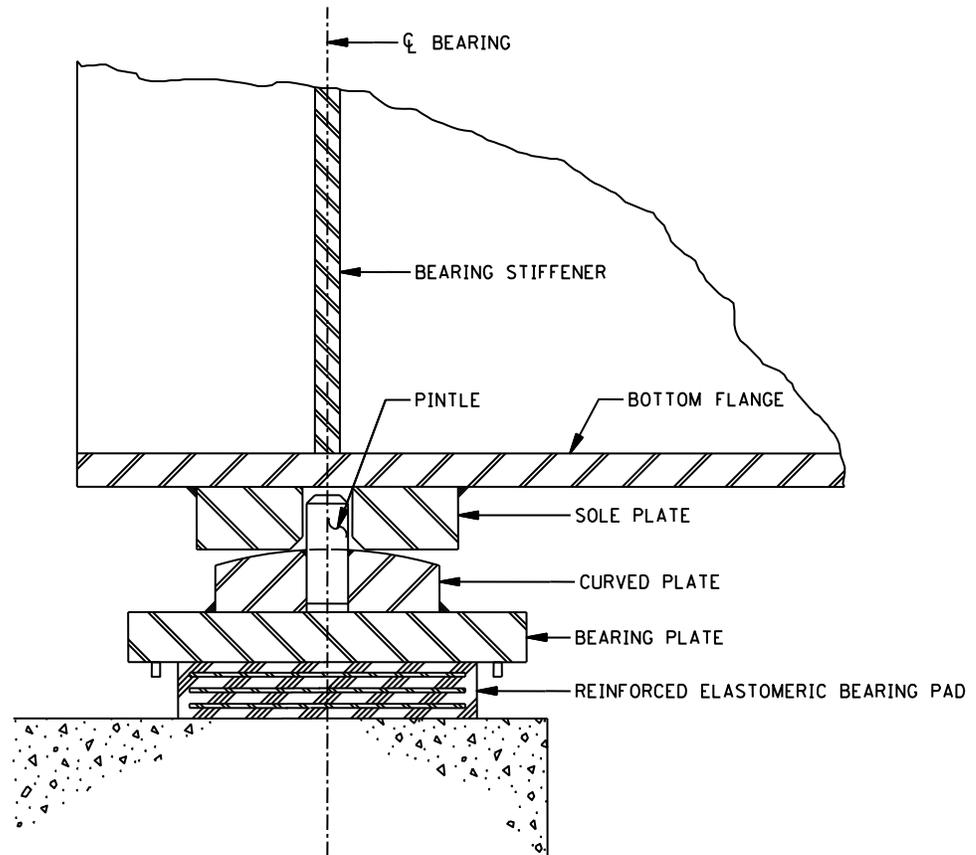


Figure 14.8.1.5

**14.8.2 Expansion  
Elastomeric  
Bearing Design  
Example  
[14.7.6]**

This example illustrates the design of an expansion curved plate elastomeric bearing for a steel plate girder bridge. The bearing is based on Bridge Details Part I, B355. The elastomeric bearing pad is designed using Method A (LRFD Article 14.7.6). Figure 14.8.2.1 labels the primary components for this type of bearing.



CURVED PLATE EXPANSION BEARING COMPONENTS

SEE DETAIL B355

**Figure 14.8.2.1**

After the maximum reaction is calculated, the bearing design should be selected from standard bearing tables in Article 14.7 of this manual. If a standard design will not work due to unusual loads or geometric constraints, a custom design will be required.

This example will outline the procedure to custom design an expansion elastomeric bearing. First determine the size of the pad required. Next determine the steel plate requirements for the rest of the assembly.

Two movements are accommodated with this type of bearing, rotation and horizontal translation. The rotation takes place at the interface

between the sole plate and the curved plate. The horizontal translation takes place in the reinforced elastomeric bearing pad.

**A. Design  
Reinforced  
Elastomeric  
Bearing Pad**

The bearing pad needs sufficient plan area to ensure that compression stresses are below the limit. It also needs sufficient thickness to accommodate the horizontal translation. Begin by determining the design movements and loads for the bearing.

**Design Movements**

The bearing is located at the abutment of a two-span steel plate girder bridge with equal spans of 152'-0". Fixity is assumed at the pier.

**[6.4.1]**

Expansion length =  $L_{exp} = 152$  ft

Coefficient of thermal expansion for steel =  $\alpha_{steel} = 0.0000065$

Design temperatures:

Base Construction Temperature:  $T_{constr} = 45$  °F

Low Temperature:  $T_{low} = -30$  °F

High Temperature:  $T_{high} = 120$  °F

Temperature Fall:  $T_{fall} = T_{constr} - T_{low} = 75$  °F

Temperature Rise:  $T_{rise} = T_{high} - T_{constr} = 75$  °F

Movement for minimum compressive stress check (Load Factor = 1.0)

$$\Delta_u = 1.0 \cdot T_{fall} \cdot \alpha_{steel} \cdot L_{exp} = 1.0 \cdot 75 \cdot 0.0000065 \cdot 152 \cdot 12 = 0.89 \text{ in}$$

Movement for shear deformation check (Load Factor = 1.3)

$$\Delta_s = 1.3 \cdot T_{fall} \cdot \alpha_{steel} \cdot L_{exp} = 1.3 \cdot \Delta_u = 1.3 \cdot 0.89 = 1.16 \text{ in}$$

**Design Loads**

The design loads for the bearing are given as follows:

Dead load =  $P_{dl} = 117$  kips

Maximum live load (without IM) =  $P_{llmax} = 108$  kips

Minimum live load (without IM) =  $P_{llmin} = -15$  kips

The bearing is sized using the maximum service limit state load:

$$P_{smax} = P_{dl} + P_{llmax} = 117 + 108 = 225 \text{ kips}$$

The minimum compressive load check is made with Strength I limit state load:

$$P_{umin} = 0.9 \cdot P_{dl} + 1.75 \cdot P_{llmin} = 0.9 \cdot 117 + 1.75 \cdot (-15) = 79.1 \text{ kips}$$

**Size Elastomeric Bearing Pad***[Table 14.7.6.2-1]*

MnDOT Spec. 3741 specifies an elastomeric pad with a hardness of 60 durometers. Per LRFD Table 14.7.6.2-1, the shear modulus  $G$  for design ranges from 0.130 to 0.200 ksi.

In order to accommodate shear deformation in the pad due to thermal movement, the total thickness of elastomer must be at least twice the design movement. The movement  $\Delta_s$  with the 1.3 multiplier is used for this check.

*[Eq. 14.7.6.3.4-1]*

$$\text{Minimum } h_{rt} = 2 \cdot \Delta_s = 2 \cdot 1.16 = 2.32 \text{ in}$$

Thickness of cover elastomer laminate,  $h_{cover} = 0.25 \text{ in}$

Try an internal elastomer laminate thickness,  $h_{ri} = 0.375 \text{ in}$

Thickness of steel plates,  $h_s = 0.125 \text{ in}$

Determine the number of internal laminates,  $n$ , required:

$$n = \frac{\text{Min } h_{rt} - 2 \cdot h_{cover}}{h_{ri}} = \frac{2.32 - 2 \cdot 0.25}{0.375} = 4.85$$

Use 5 internal laminates.

Number of steel plates,  $n_s = n + 1 = 6$

Total elastomer thickness:

$$h_{rt} = 2 \cdot (h_{cover}) + n \cdot (h_{ri}) = 2 \cdot (0.25) + 5 \cdot (0.375) = 2.375 \text{ in}$$

$$\begin{aligned} \text{Height of reinforced elastomeric pad, } D &= h_{rt} + n_s \cdot h_s \\ &= 2.375 + 6 \cdot 0.125 = 3.125 \text{ in} \end{aligned}$$

For preliminary pad sizing, assume the pad allowable compression is 1.25 ksi. Round the pad width and length dimensions to even inch dimensions. For steel beams, the width ( $B$ ) must be at least the bottom flange width and not more than 2 inches greater than the bottom flange width. In this case, the bottom flange width is 20 inches.

Try a pad width,  $B = 20 \text{ in}$

Solve for the minimum pad length (A):

$$A_{\min} = \frac{P_{s\max}}{1.25 \cdot B} = \frac{225}{1.25 \cdot 20} = 9.00 \text{ in}$$

Try a pad length, A = 10 in

### Shape Factor Check

Check the shape factor of the internal laminate:

$$S_i = \frac{A \cdot B}{2 \cdot (A+B) \cdot h_{ri}} = \frac{10 \cdot 20}{2 \cdot (10+20) \cdot 0.375} = 8.89$$

$$5.0 \leq S_i = 8.89 \leq 10.0 \quad \underline{\text{OK}}$$

#### [14.7.6.1]

Check that shape factor requirements of AASHTO LRFD Art. 14.7.6.1 are met:

For this check, if cover layer thickness is greater than or equal to internal laminate thickness, n may be increased 0.5 for each cover layer.

Then

$$\frac{S_i^2}{n} = \frac{8.89^2}{5+0.5+0.5} = 13.17 < 22 \quad \underline{\text{OK}}$$

Compute the shape for the cover layers for later use in the deflection computations.

#### [14.7.5.1]

$$S_c = \frac{A \cdot B}{2 \cdot (A+B) \cdot h_{ri}} = \frac{10 \cdot 20}{2 \cdot (10+20) \cdot 0.25} = 13.33$$

### Pad Dimensional Checks

Check that the bearing satisfies aspect ratio checks. The total elastomeric thickness,  $h_{rt}$ , must be less than  $1/3$  the length of the pad's shortest side.

#### [14.7.6.3.6]

$$\frac{A}{3} = \frac{10}{3} = 3.33 \text{ in} > 2.375 \text{ in} \quad \underline{\text{OK}}$$

Also check that maximum pad dimension (B) is no greater than 2.5 times the smallest pad dimension (A):

$$2.5 \cdot A = 2.5 \cdot 10 = 25 \text{ in} > 20 \text{ in} \quad \underline{\text{OK}}$$

**[14.7.6.3.2]****Maximum Compressive Stress Check**

Now check the maximum compressive stress in the pad. Use the minimum shear modulus for this computation ( $G_{\min} = 0.130$  ksi).

The allowable compressive stress  $\sigma_{\text{sall}}$  is the smaller of:

$$\begin{aligned}\sigma_{\text{sall}} &= 1.25 \cdot G_{\min} \cdot S_i \\ &= 1.25 \cdot 0.130 \cdot 8.89 \\ &= 1.44 \text{ ksi}\end{aligned}$$

or  $\sigma_{\text{sall}} = 1.25$  ksi. GOVERNS

Then the maximum service limit state stress is:

$$\text{Actual } \sigma_s = \frac{P_s}{A \cdot B} = \frac{225}{10 \cdot 20} = 1.13 \text{ ksi} < 1.25 \text{ ksi} \quad \text{OK}$$

**[14.7.6.3.3]****Compressive Deflection****[14.7.5.3.6]**

To ensure that joints and appurtenances perform properly, the vertical deflection in elastomeric bearings is checked. Due to the nonlinear behavior of the elastomer, the movement associated with live load is computed by subtracting the dead load deflection from the total load deflection.

Begin by determining the average vertical compressive stress in the bearings under dead load alone and under total load.

$$\sigma_{\text{dl}} = \frac{P_{\text{dl}}}{A \cdot B} = \frac{117}{10 \cdot 20} = 0.585 \text{ ksi}$$

$$\sigma_{\text{tl}} = \frac{P_{\text{tl}}}{A \cdot B} = \frac{225}{10 \cdot 20} = 1.125 \text{ ksi}$$

Using the stress strain figure for 60 durometer reinforced bearings shown in AASHTO LRFD Figure C14.7.6.3.3-1, the strain was estimated in the interior laminates and the cover layers and summarized in Table 14.8.2.1.

**Table 14.8.2.1 Estimated Strains**

Laminate	Load	S	Stress (ksi)	Estimated Compressive Strain $\epsilon$ (%)
Interior	Dead Load	8.89	0.585	2.6%
	Total Load	8.89	1.125	4.2%
Cover	Dead Load	13.33	0.585	2.2%
	Total Load	13.33	1.125	3.7%

The initial compressive deflection of a single interior laminate under total load is:

**[14.7.6.3.3]**

$$\Delta_{t\text{hri}} = \epsilon \cdot h_{ri} = 0.042 \cdot h_{ri} < 0.090 \cdot h_{ri} \quad \text{OK}$$

With five interior laminates and two cover layers the deflection under total load is:

$$\begin{aligned} \Delta_{tl} &= 5 \cdot \epsilon_{ri} \cdot h_{ri} + 2 \cdot \epsilon_{cover} \cdot h_{rcover} \\ &= 5 \cdot 0.042 \cdot 0.375 + 2 \cdot 0.037 \cdot 0.25 = 0.097 \text{ in} \end{aligned}$$

The deflection under dead load is:

$$\begin{aligned} \Delta_{dl} &= 5 \cdot \epsilon_{ri} \cdot h_{ri} + 2 \cdot \epsilon_{cover} \cdot h_{rcover} \\ &= 5 \cdot 0.026 \cdot 0.375 + 2 \cdot 0.022 \cdot 0.25 = 0.060 \text{ in} \end{aligned}$$

**[Table 14.7.6.2-1]**

The deflection due to creep is:

$$\Delta_{cr} = 0.35 \cdot \Delta_{dl} = 0.35 \cdot 0.060 = 0.021 \text{ in}$$

**[C14.7.5.3.6]**

The difference between the two deflections is the estimated live load deflection. The total deflection due to live load plus creep should be no greater than  $\frac{1}{8}$  inch.

$$\Delta_{ll} = \Delta_{tl} - \Delta_{dl} = 0.097 - 0.060 = 0.037 \text{ in}$$

$$\Delta_{ll} + \Delta_{cr} = 0.037 + 0.021 = 0.058 \text{ in} < 0.125 \text{ in} \quad \text{OK}$$

**Minimum Compressive Load Check**

Using the equation derived in BDM Article 14.3.3.3.1:

$$\begin{aligned} \text{Req'd. } P_{\text{umin}} &\geq 5 \cdot G_{\text{max}} \cdot A_{\text{pad}} \cdot \frac{\Delta_u}{h_{\text{rt}}} \\ &= 5 \cdot 0.200 \cdot 10 \cdot 20 \cdot \frac{0.89}{2.375} = 74.9 \text{ kips} \end{aligned}$$

$$\text{Actual } P_{\text{umin}} = 79.1 \text{ kips} > 74.9 \text{ kips} \quad \underline{\text{OK}}$$

**[14.7.5.3.5]****Check Service and Fatigue of Steel Reinforcement Plates**

Check the service and fatigue limit states for the steel plates. At the service limit state the following equation must be satisfied:

$$h_s \geq \frac{3 \cdot h_{\text{max}} \cdot \sigma_s}{F_y}$$

The yield strength of the steel plates ( $F_y$ ) is 36 ksi.

$$h_{\text{max}} = h_{\text{ri}} = 0.375 \text{ in}$$

$$\sigma_s = 1.13 \text{ ksi}$$

$$\text{Min. } h_s = \frac{3 \cdot h_{\text{max}} \cdot \sigma_s}{F_y} = \frac{3 \cdot 0.375 \cdot 1.13}{36} = 0.035 \text{ in} < 0.125 \text{ in} \quad \underline{\text{OK}}$$

At the fatigue limit state, the following equation must be satisfied:

$$h_s \geq \frac{2 \cdot h_{\text{max}} \cdot \sigma_L}{\Delta_{\text{FTH}}}$$

**[Table 6.6.1.2.5-3]** where,  $\Delta_{\text{FTH}} = 24$  ksi (Category A steel detail).

Note that the live load used for this check is not based on reactions from the fatigue truck and is not factored according to the fatigue limit state. Rather, it is the maximum live load for the service limit state with a load factor equal to 1.0.

$$\sigma_L = \frac{P_{\text{llmax}}}{A \cdot B} = \frac{108}{10 \cdot 20} = 0.540 \text{ ksi}$$

Minimum steel plate thickness for this check is

$$\text{Min. } h_s = \frac{2 \cdot h_{\text{max}} \cdot \sigma_L}{\Delta_{\text{FTH}}} = \frac{2 \cdot 0.375 \cdot 0.540}{24} = 0.017 < 0.125 \text{ in} \quad \underline{\text{OK}}$$

Use a 10" x 20" x 3<sup>1</sup>/<sub>8</sub>" bearing pad, composed of two 1/4 inch cover laminates, five 3/8 inch interior laminates, and six 1/8 inch steel plates.

**B. Curved Plate Design**

The thickness of the plate is H. The curved plate has a width (B), which is equal to the width of the bearing pad. The length (G) is determined in an iterative process with the thickness. Begin by checking the thickness for a curved composite plate with a length of 4.5 inches. If, when designing the bearing plate, the required bearing plate thickness exceeds 2 inches, increase the length of the curved plate to reduce the length of the cantilever for the bearing plate design. Increase the curved plate length until the required bearing plate thickness alone and the required composite plate thickness for the curved plate design become approximately equal.

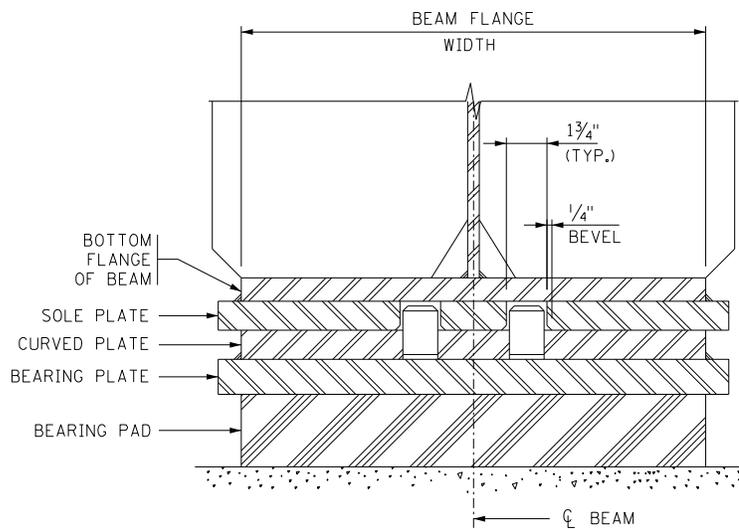
Try a 20" x 4.5" curved plate (B = 20 in, G = 4.5 in).

First, determine the radius of the contact surface. The radius of the curved plate is a function of the yield strength of the steel and the load intensity.

The contact length of the sole plate with the curved plate is equal to the curved plate width minus the pintles and bevels. Refer to Figure 14.8.2.2.

Contact length  $L_{sp}$  is equal to

$$L_{sp} = 20 - 2 \cdot (1.75) - 2 \cdot (0.25) - 2 \cdot (0.25) = 15.50 \text{ in}$$



**Figure 14.8.2.2**

**[14.7.1.4]**

Based on past satisfactory performance of curved plate bearing assemblies, the minimum radius permitted is determined with LRFD Equation C14.7.1.4-1 and C14.7.1.4-2. Start by assuming the diameter  $d$  is 25 inches or less, so use the first equation. Rearranging the equation to solve for diameter results in the following:

$$d_{\min} = \frac{20 \cdot p}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left( \frac{P_s}{L_{sp}} \right)}{0.6 \cdot (F_y - 13)} = \frac{20 \cdot \left( \frac{225}{15.50} \right)}{0.6 \cdot (36 - 13)} = 21.0 \text{ in} < 25.0 \text{ in}$$

The assumption was correct. Then the radius  $R_{\min} = 10.5$  in

The radius of curved plates is to be no less than 16 inches. Therefore, specify the minimum radius for the curved plate to be 16 inches.

Use strength limit state loads for flexural design of the curved plate.

The maximum Strength I limit state load,  $P_{u\max}$ , is:

$$\begin{aligned} P_{u\max} &= 1.25 \cdot P_{dl} + 1.75 \cdot P_{ll\max} \\ &= 1.25 \cdot 117 + 1.75 \cdot (108) = 335.3 \text{ kips} \end{aligned}$$

Pressure on the composite plate is:

$$\sigma_{cp} = \frac{P_{u\max}}{G \cdot B} = \frac{335.3}{4.5 \cdot 20} = 3.73 \text{ ksi}$$

Maximum moment on the composite plate is:

$$M_{ucp} = \sigma_{cp} \cdot \frac{G}{2} \cdot \frac{G}{4} \cdot B = 3.73 \cdot \frac{4.5}{2} \cdot \frac{4.5}{4} \cdot 20 = 188.8 \text{ kip-in}$$

**[6.12.2.2.7]**

Consider the plate to be fully laterally supported. The AASHTO LRFD Specifications allow the nominal flexural resistance of a rectangular section to be taken as the plastic moment. However, MnDOT limits the nominal resistance to the yield moment. Find the required composite plate thickness such that the yield moment,  $M_y$ , of the section will have adequate capacity to resist the design moment,  $M_{ucp}$ .

**[6.5.4.2]**

For steel elements in flexure,  $\phi_f = 1.0$ .

The flexural resistance,  $M_r$ , of the composite plate section is:

$$M_r = \phi_f \cdot M_{ncp} = \phi_f \cdot M_y = \phi_f \cdot S_{cp} \cdot F_y$$

The section modulus of the composite plate is:

$$S_{cp} = \frac{B \cdot H^2}{6}$$

where H = thickness of composite plate

Then by substitution:

$$M_r = \phi_f \cdot \left( \frac{B \cdot H^2}{6} \right) \cdot F_y$$

Set the flexural resistance of the composite plate section equal to the design moment:

$$M_{ucp} = \phi_f \cdot \left( \frac{B \cdot H^2}{6} \right) \cdot F_y$$

Solve for composite plate thickness:

$$H \geq \sqrt{\frac{6 \cdot M_{ucp}}{\phi_f \cdot F_y \cdot B}} = \sqrt{\frac{6 \cdot 188.8}{1.0 \cdot 36 \cdot 20}} = 1.25 \text{ in}$$

The standard curved plate thickness is 1¼ inches, so composite action does not need to be considered. Use a 1¼ thick curved plate.

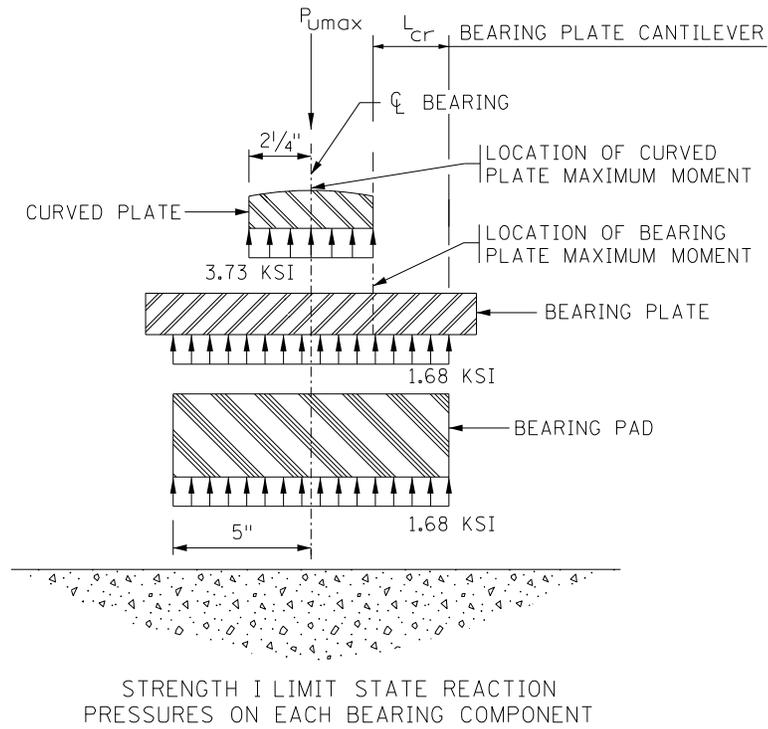


Figure 14.8.2.3

**C. Bearing Plate Design**

Now determine the thickness of the bearing plate. The bearing plate has plan dimensions that are slightly larger than the bearing pad to provide adequate space for the attachment of the keeper bar. One inch is provided on all sides for this purpose.

Bearing Plate width,  $E = 22$  in

Bearing Plate length,  $C = 12$  in

The bearing plate is assumed to act as a cantilever that carries the bearing pad pressure back to the curved plate. See Figure 14.8.2.3.

The cantilever length is half the difference in length between the bearing pad and the curved plate.

$$\sigma_{bp} = \frac{P_{umax}}{A \cdot B} = \frac{335.3}{10 \cdot 20} = 1.68 \text{ ksi}$$

$$L_{cr} = \frac{A}{2} - \frac{G}{2} = \frac{10}{2} - \frac{4.5}{2} = 2.75 \text{ in}$$

$$M_{ubp} = \sigma_{bp} \cdot \frac{L_{cr}^2}{2} \cdot E = 1.68 \cdot \frac{2.75^2}{2} \cdot 22 = 139.8 \text{ kip-in}$$

Again, the AASHTO LRFD specifications allow the nominal flexural capacity to be set equal to the plastic moment of the plate. However, MnDOT limits the nominal capacity of the plate to the yield moment. Find the required bearing plate thickness such that the yield moment,  $M_y$ , of the section will have adequate capacity to resist the design moment,  $M_{ubp}$ .

The flexural resistance,  $M_r$ , of the bearing plate section is:

$$M_r = \phi_f \cdot M_{nbp} = \phi_f \cdot M_y = \phi_f \cdot S_{bp} \cdot F_y$$

The section modulus of the bearing plate is:

$$S_{bp} = \frac{E \cdot F^2}{6}$$

where  $F$  = thickness of bearing plate

Then by substitution:

$$M_r = \phi_f \cdot \left( \frac{E \cdot F^2}{6} \right) \cdot F_y$$

Set the flexural resistance of the bearing plate section equal to the design moment:

$$M_{ubp} = \phi_f \cdot \left( \frac{E \cdot F^2}{6} \right) \cdot F_y$$

Solve for bearing plate thickness:

$$F \geq \sqrt{\frac{6 \cdot M_{ubp}}{\phi_f \cdot F_y \cdot E}} = \sqrt{\frac{6 \cdot 139.8}{1.0 \cdot 36 \cdot 22}} = 1.03 \text{ in}$$

The standard bearing plate thickness is 1½ inches, so use a 1½ thick bearing plate.

#### ***D. Sole Plate Constraints***

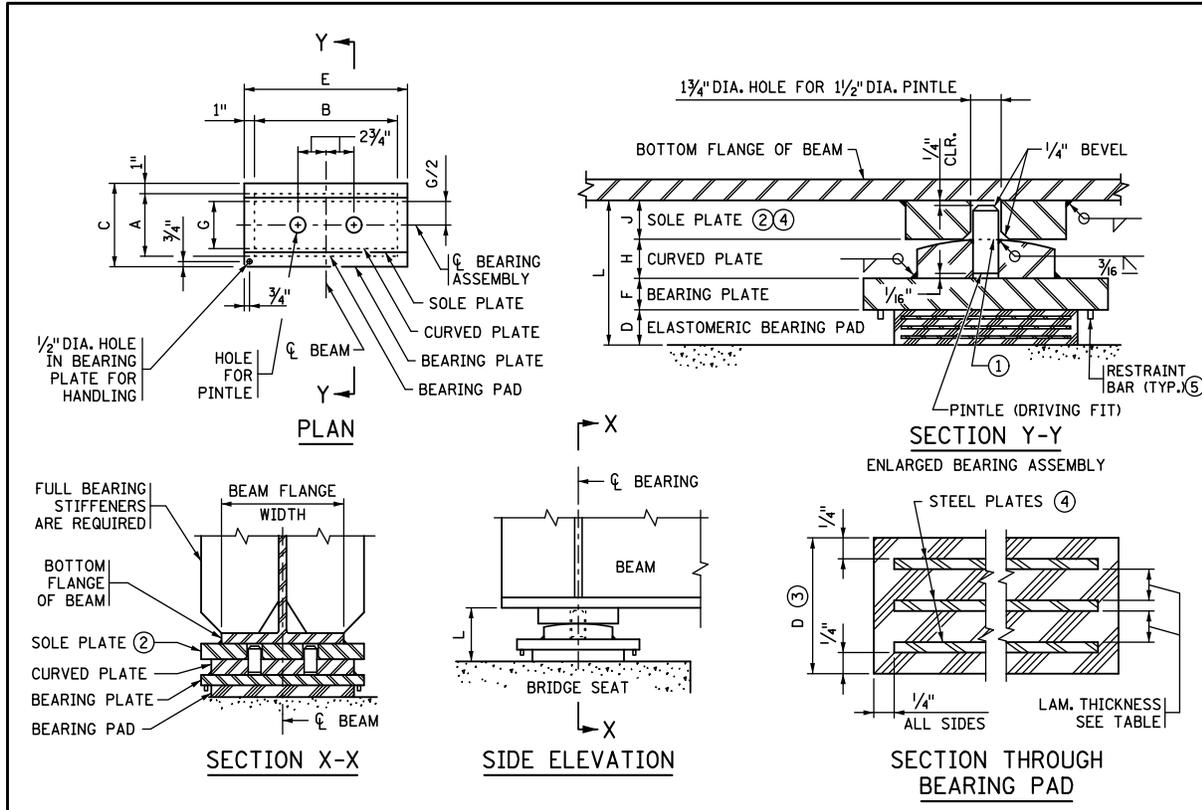
Set the sole plate width 2 inches greater than the curved plate width and check that it is sufficiently wider than the beam bottom flange to allow welding.

$$\text{Sole plate width} = 20 + 2 = 22 \text{ in} > 20 \text{ in flange} \quad \underline{\text{OK}}$$

The sole plate length must be 6 inches minimum, but not less than the curved plate length. Therefore, set sole plate length equal to 6 inches.

The minimum sole plate thickness is 1¼ inches. When the bearing pad width exceeds the bottom flange width, the sole plate must be designed as a cantilever to resist the load from the pad that extends outside the flange. For our case, the bottom flange width equals the pad width, so set sole plate thickness equal to 1¼ inches.

The bearing design is summarized in Figures 14.8.2.4 and 14.8.2.5.



TABLE

ASSEMBLY TYPE	LOCATION	BEAM FLANGE WIDTH	BEARING PAD SIZE			STEEL PLATES		LAMINATES	SHAPE FACTOR	BEARING PLATE SIZE			CURVED PLATE SIZE			SOLE PLATE SIZE		PINTLE DIA.	ASSY. HEIGHT L	RESTRAINT PATTERN ⑤			
			A	B	D	NO.	THICK.			NO.	THICK.	C	E	F	G	B	H				R ①	WID.	LEN. J ②
E	ABUT.	20"	10"	20"	3/8"	6	1/8"	5	3/8"	8.89	12"	22"	1 1/2"	4 1/2"	20"	1 1/4"	16"	22"	6"	1 1/4"	1 1/2"	7 1/8"	A-3

**NOTES:**

PROVIDE ELASTOMERIC MATERIALS AND PAD CONSTRUCTION PER SPEC. 3741.

PROVIDE STEEL PLATES PER SPEC. 3306 EXCEPT THE SOLE PLATE. PROVIDE SOLE PLATE WITH THE SAME MATERIAL SPECIFICATION AS THE STEEL BEAMS.

PROVIDE PINTLES PER SPEC. 3309.

GALVANIZE STRUCTURAL STEEL BEARING ASSEMBLY AFTER FABRICATION PER SPEC. 3394, EXCEPT AS NOTED.

PAYMENT FOR BEARING ASSEMBLY INCLUDES ALL MATERIAL ON THIS DETAIL EXCEPT THE SOLE PLATE. THE SOLE PLATE IS INCLUDED IN THE WEIGHT OF STRUCTURAL STEEL.

① THE MIN. RADIUS IS 16" UNLESS OTHERWISE SPECIFIED IN THE TABLE. THE MAX. RADIUS IS 24". FINISH TO 250 MICRO. THE FINISHED THICKNESS OF THE PLATE MAY BE 1/16" LESS THAN SHOWN.

② WHEN THE SOLE PLATE IS TAPERED, DIMENSIONS "J" AND "L" ARE THICKNESS OF SOLE PLATE AND BEARING ASSEMBLY AT CENTERLINE OF BEARING.

③ THE TOTAL THICKNESS SHOWN INCLUDES THE STEEL PLATES.

④ DO NOT GALVANIZE THIS PLATE.

⑤ REFER TO BEARING PAD RESTRAINT SHEET FOR ADDITIONAL INFORMATION AND DETAILS.

**DESIGN DATA:**

MAX. FACTORED SHEAR RESISTANCE:  
- 50.3 KIPS PER 1/2" DIA. PINTLE

APPROVED: NOVEMBER 22, 2002

*Daniel J. Hanson*  
STATE BRIDGE ENGINEER

STATE OF MINNESOTA  
DEPARTMENT OF TRANSPORTATION

CURVED PLATE BEARING ASSEMBLY  
(STEEL BEAMS)  
(EXPANSION)

REVISED  
08-10-2006  
12-17-2008  
11-03-2015  
11-02-2017

DETAIL NO.

B355

Figure 14.8.2.4

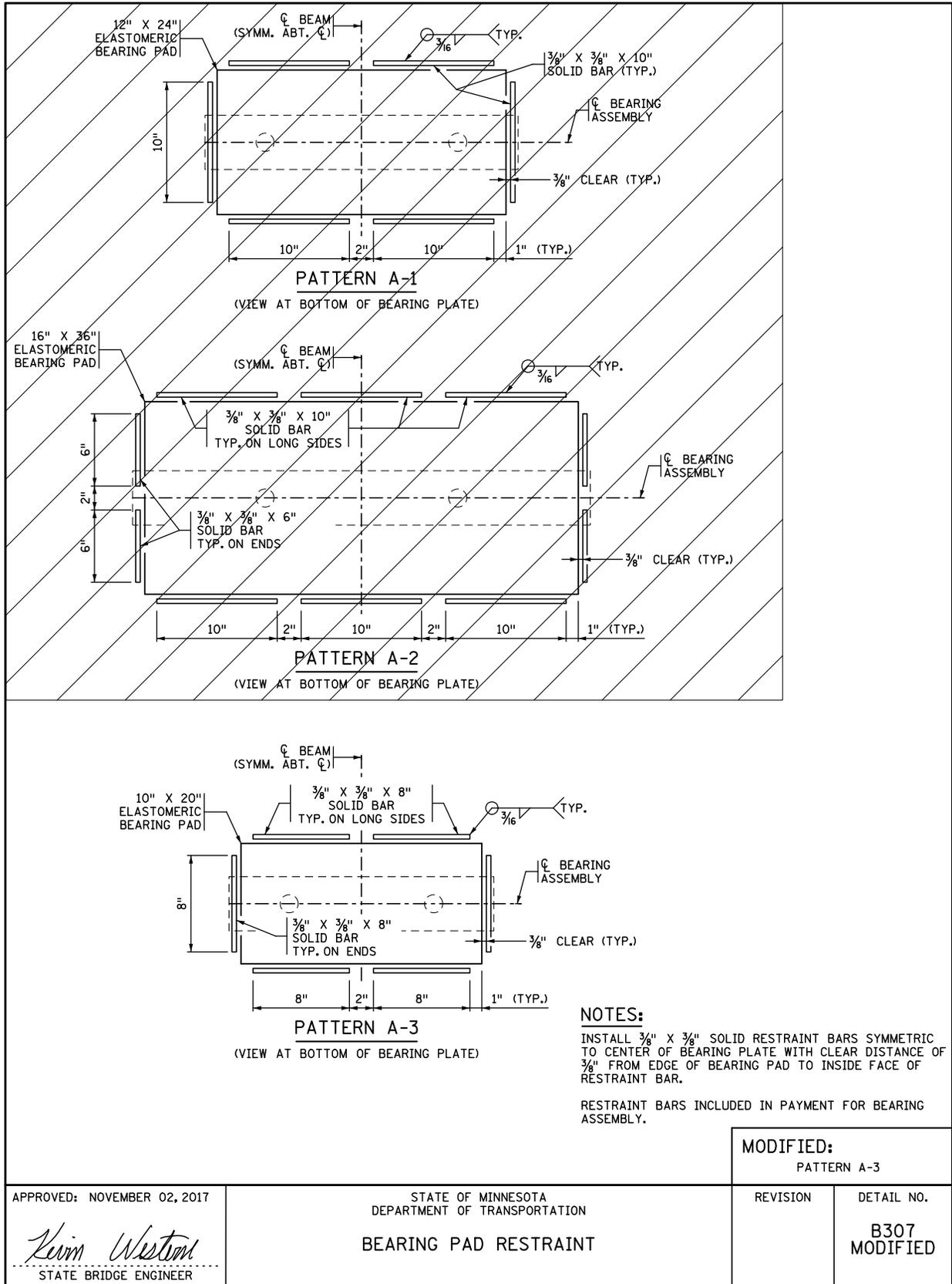


Figure 14.8.2.5



Minnesota Department of Transportation

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## Memo

Bridge Office  
Mail Stop 610  
3485 Hadley Ave.  
Oakdale, MN 55128

Office Tel:(651) 366-4506  
Fax: 366-4497

October 3, 2007

To: Bridge Design Engineers

From: Kevin Western *Kevin Western*  
Bridge Design Engineer

### **MEMO TO DESIGNERS (2007-02): Adhesive Anchors under Sustained Tensile Loads**

Following the failure of adhesive anchors that held roof panels on the Boston Arterial Project, NTSB issued an advisory regarding the use of adhesive anchorages in sustained tensile overhead applications. There may be other installation orientations other than overhead where adhesive anchors are in sustained tension. Attached below is the MnDOT Memo prohibiting the use of adhesive anchors in a sustained tensile load installation. The NTSB advisory can be viewed at: <http://ntsb.gov/Publictn/2007/HAR-07-02.htm>

Until specifications are developed by ACI or AASHTO concerning long term creep characteristics in adhesive anchors, use of adhesive anchors in sustained tension is prohibited. Other applications utilizing adhesive anchors, such as metal rail attachment, in a non-direct tensile application is permitted.

cc: D. Dorgan  
D. Dahlberg  
M. Louis  
E. Wolhowe / Design Consultants



## Minnesota Department of Transportation

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### Memo

September 24, 2007

TO: OPSDiv, PCMG, CMG, AMEs and Office Directors

FROM: Richard Arnebeck, Director   
Engineering Services Division

Robert Winter, Director   
Operations Division

SUBJECT: National Transportation Safety Board (NTSB) – Safety Recommendation  
Adhesive Anchoring Systems in Highway Construction  
NTSB Ref. Numbers: H-07-23 and H-07-24

The NTSB has completed its investigation of the July 10, 2006 ceiling collapse in the Interstate 90 Connector Tunnel in Boston, Massachusetts. As a result of the investigation, the NTSB has issued a Safety Recommendation concerning the use of Adhesive Anchoring Systems in Highway Construction:

The NTSB has made the following recommendations to the Minnesota Department of Transportation:

“Prohibit the use of adhesive anchors in sustained tensile-load overhead highway applications where failure of the adhesive would result in a risk to the public until testing standards and protocols have been developed and implemented that ensure the safety of these applications (H-07-23).”

“Review the use of adhesive anchors in highway construction within your jurisdiction and identify those sites where failure of the adhesive under sustained load could result in a risk to the public. Once those sites have been identified, implement an inspection and repair program to ensure that such failures do not occur (H-07-24).”

**Effective immediately, Mn/DOT is prohibiting the use of adhesive anchors in all sustained tensile-load applications, until testing standards and protocols have been developed and implemented to ensure the safety of these applications.**

Mn/DOT is currently in the process of reviewing and identifying all uses and applications of adhesive anchors under sustained loads within its jurisdiction. An inspection and repair program will be implemented once the full extent of the issue is known. We expect the implementation plan to be in place by November 3, 2007.

Please ensure that all your staff involved in this matter has a copy of this directive.

If you have any questions or concerns, please contact Mukhtar Thakur at 651/366-4691.  
Thank you.



# Memo

Bridge Office  
Mail Stop 610  
3485 Hadley Ave.  
Oakdale, MN 55128

Office Tel: (651)366-4506  
Fax: 366-4497

September 18, 2008

To: Bridge Design Engineers  
From: Kevin Western *Kevin Western*  
Bridge Design Engineer

## MEMO TO DESIGNERS (2008-01): Prestressed Concrete Design – Calculation of Prestress Losses and Beam Camber & Deflection

The prestress loss provisions adopted in the 2005 interims of the AASHTO LRFD Bridge Design Specifications provide loss estimates that are substantially lower than those of the previous provisions. A review by the Mn/DOT Bridge Office has resulted in the following policy for calculation of prestress losses:

- 1) Calculate the short term losses (elastic shortening losses) in accordance with AASHTO LRFD Article 5.9.5.2.3a or 5.9.5.2.3b as appropriate. Do not transform the strands. Also, do not include any elastic gains due to the application of loads.
- 2) For pretensioned I-beam and rectangular beam sections, calculate the time-dependent losses using the approximate method of AASHTO LRFD Article 5.9.5.3. For post-tensioned slabs, calculate the time-dependent losses using the refined method of AASHTO LRFD Article 5.9.5.4.
- 3) For bridges where live load distribution has been determined using the approximate methods of AASHTO LRFD Articles 4.6.2.2 or 4.6.2.3, check the tensile stress limits given in AASHTO LRFD Article 5.9.4 using the Service III load combination. For bridges where live load distribution has been determined using a refined analysis, check the tensile stress limits using the Service I load combination.
- 4) Use the table shown in Figure 1 below to report the prestress losses in the bridge plan. Until the standard sheets are revised, add the table to the prestressed beam sheet in the bridge plan and mark the sheet "Modified". Load rating is currently done using VIRTIS, which calculates losses using the AASHTO Standard Specifications. Including the losses in the bridge plan will provide a permanent record from which the load rating engineer can manually input the losses into VIRTIS to get a load rating more consistent with the design.

CALCULATED PRESTRESS LOSSES	
ELASTIC SHORTENING LOSS	_____ KSI
LONG TERM LOSSES	_____ KSI

FIGURE 1

Accomplishment of the above policy using commercial computer programs requires use of correct input and settings within the programs. For example, the current version of CONSPAN does not allow the option of ignoring elastic gains due to the application of loads. Therefore the losses must be entered manually. For questions about computer programs, contact Khalid Obeidat at [khalid.obeidat@dot.state.mn.us](mailto:khalid.obeidat@dot.state.mn.us) or by phone at 651-366-4485.

Calculation of prestressed beam camber and deflection in accordance with the *MEMO TO DESIGNERS (2007-03)* has caused some confusion in regards to computer programs and transforming of the strands. Therefore, the *MEMO TO DESIGNERS (2007-03)* is hereby deleted and is superceded by the following:

- 1) Complete the prestressed beam design using one of the available programs without transforming the strands. Change both the camber and deflection multipliers at erection from the default values of 1.8 and 1.85 to a value of 1.5.
- 2) Report the values determined above on the prestressed concrete beam sheet in the bridge plan.

If you have any questions or concerns about the policy, please contact Dave Dahlberg at [dave.dahlberg@dot.state.mn.us](mailto:dave.dahlberg@dot.state.mn.us) or by phone at 651-366-4491.

cc: D. Dorgan  
D. Dahlberg  
E. Wolhowe / Design Consultants  
G. Peterson  
T. Styrbicki



## Memo

**Bridge Office**  
Mail Stop 610  
3485 Hadley Ave.  
Oakdale, MN 55128

Office Tel: (651)366-4506  
Fax: 366-4497

October 20, 2008

To: Bridge Design Engineers

From: Kevin Western  
Bridge Design Engineer

A handwritten signature in blue ink that reads "Kevin Western".

### **MEMO TO DESIGNERS (2008-02): Truss Bridge Gusset Plate Analysis**

The collapse of the I35W bridge over the Mississippi River sparked a structural review of gusset plates in truss bridges throughout the state. In order to accomplish this task, a procedure for analyzing and rating the gusset plates was developed. This has resulted in the following Mn/DOT Bridge Office policy for analysis and rating of truss bridge gusset plates:

- 1) Mn/DOT Truss Bridge Gusset Plate Design Review Procedure (see attachment)  
This document details the gusset plate checks to be made for determination of inventory and operating ratings.
- 2) Mn/DOT Interpretation of Truss Bridge Gusset Plate Review Results (see attachment)  
This document describes how the gusset plate review results are to be interpreted and also details refined analysis methods for buckling and shear checks.

This Memo to Designers is being published now in order to make it official policy while structural review of the truss bridges on the local system is occurring. A comprehensive section on bridge rating is currently being developed for insertion in the LRFD Bridge Design Manual and will include the guidance contained in this memo when complete.

If you have any questions or concerns about the policy, please contact Dave Dahlberg at [dave.dahlberg@dot.state.mn.us](mailto:dave.dahlberg@dot.state.mn.us) or by phone at 651-366-4491.

cc: D. Dorgan  
D. Dahlberg  
E. Wolhowe / Design Consultants  
G. Peterson  
T. Styrbicki  
R. Garcia (FHWA)

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## ***Mn/DOT Truss Bridge Gusset Plate Design Review Procedure***

- 1) Gather information and determine loads for gusset plate review.
  - a. Obtain the original bridge plan, any subsequent repair plans, the shop drawings, and the most current inspection report.
  - b. Verify (by approximate methods or frame analysis) the original dead load from the original plan truss load table (if available) and the live load where appropriate. The gusset plates will be rated for an HS20 loading. Note that original live load will likely not be HS20. Therefore, the live load will need to be increased by the ratio of HS20 to the original live load or be recalculated using an analysis program or BARS influence line data. Note that for bridges designed before 1931, the H15 loading used was not the same as H15 used today, so live load values for this case must be regenerated. Also note that the HS20 lane load concentrated load for moment (18 kips) is to be used for determination of top and bottom chord loads. The HS20 lane load concentrated load for shear (26 kips) is to be used for determination of diagonal and vertical loads. The original plan truss load tables typically include only the maximum member forces. Therefore for flexure and shear checks, coincident member live load forces must also be generated.
  - c. Determine increased dead load in truss joint members due to additional loads (overlays, railings, etc) using one of the following methods:
    - Use influence lines from BARS or original truss design calculations (if available) to calculate truss member tension/compression due to additional loads.
    - Model truss using STAAD or another analysis program to determine truss member forces due to additional loads.
  - d. Review the most current inspection report and identify gusset plates with appreciable section loss. If appreciable loss is reported, discuss with bridge inspector to determine the percentage of section loss. Otherwise, report ratings for the following scenarios:
    - For rivet/bolt shear and bearing check, report ratings for capacity reduction factors  $CRF = 1.00$  and  $CRF = 0.95$ , both with 0% section loss
    - For all other checks, report ratings for capacity reduction factors  $CRF = 0.95$  and  $CRF = 0.85$ , both with 0% section lossThe range of results will be considered upon completion of the review.
  - e. Confirm that gusset plates shown in the plans match those shown in the shop drawings in thickness and material.
  
- 2) Check tension in gusset plates.
  - a. Consider all truss joints with tension members. Use engineering judgement to eliminate checking of similar member connections.

- b. Determine “Whitmore effective width” for each member. This is done by finding the first set of rivets in the member to gusset plate connection and drawing lines that start at the outside rivets radiating at 30 degrees outward from the direction of the member. The “Whitmore effective width” is equal to the distance between the 30 degree lines where they intersect a line through the last set of rivets. See Attachment 1, taken from the *AISC Steel Construction Manual 13<sup>th</sup> Edition*.
- c. Determine factored resistance per LRFD 6.8.2.1 using “Whitmore effective width” for calculation of gusset plate gross and net area.
- d. Calculate the inventory and operating rating factor (RF) as a function of HS20 live loading:

$$RF_{inv} = (\phi \cdot P_n - 1.3 \cdot P_{DL}) / 2.17 \cdot P_{HS20LL}$$

$$RF_{op} = (\phi \cdot P_n - 1.3 \cdot P_{DL}) / 1.3 \cdot P_{HS20LL}$$

If pedestrian live load is present, include entire amount when calculating rating factors:

$$RF_{inv} = (\phi \cdot P_n - 1.3 \cdot P_{DL} - 2.17 \cdot P_{ped LL}) / 2.17 \cdot P_{HS20LL}$$

$$RF_{op} = (\phi \cdot P_n - 1.3 \cdot P_{DL} - 1.3 \cdot P_{ped LL}) / 1.3 \cdot P_{HS20LL}$$

- 3) Check flexural capacity of gusset plates.
  - a. Consider all truss joints. Use engineering judgement to eliminate checking of similar member connections.
  - b. Cut a section through the gusset plate at a line parallel to the chord that passes through the last line of rivets in diagonal/vertical members.
  - c. Determine gross section properties. Consider all plates that will resist bending at the joint being checked. This includes gusset plates as well as top, bottom, and side plates (if they exist).
  - d. Determine combined axial and bending stress ( $f = P/A + My/I$ ) on the gross section due to eccentricity of max loading to neutral axis of plates for DL and LL. Use coincident LL forces determined by analysis. For cases where the coincident live load force is greater than the maximum force determined for the member (which may happen for top or bottom chords where max forces were determined with the 18 kip concentrated load, but coincident forces were determined using the 26 kip concentrated load), limit the coincident live load force to the maximum live load force.
  - e. Determine factored resistance based on a limiting stress of  $\phi_r \cdot F_y$  per Guide Specs for Strength Design of Truss Bridges Article 1.11 and LRFD 6.14.2.8.
  - f. Determine rating factors:
 
$$RF_{inv} = (\phi_r \cdot F_y - 1.3 \cdot f_{DL}) / 2.17 \cdot f_{HS20LL}$$

$$RF_{op} = (\phi_r \cdot F_y - 1.3 \cdot f_{DL}) / 1.3 \cdot f_{HS20LL}$$
 If pedestrian live load is present, include entire amount when calculating rating factors:
 
$$RF_{inv} = (\phi_r \cdot F_y - 1.3 \cdot f_{DL} - 2.17 \cdot f_{ped LL}) / 2.17 \cdot f_{HS20LL}$$

$$RF_{op} = (\phi_r \cdot F_y - 1.3 \cdot f_{DL} - 1.3 \cdot f_{ped LL}) / 1.3 \cdot f_{HS20LL}$$
  - g. Cut a section through the gusset plate at a line normal to the chord that passes through the centerline of the joint and repeat steps above.

- h. Consider cutting sections through the gusset plate at other locations in order to maximize the combined axial and bending stress and repeat steps above.
- 4) Check shear in gusset plates.
- Consider all truss joints. Use engineering judgement to eliminate checking of similar member connections.
  - Cut a section through the gusset at a line parallel to the chord that passes through the line of rivets in the chord closest to the diagonal/vertical members.
  - Determine shear for DL and LL at the cut section. Use coincident LL forces determined by analysis.
  - Determine both gross area and net area of the plates at the cut section.
  - Determine factored yield resistance  $\phi \cdot V_{ny} = \phi_{vy} \cdot \Omega \cdot (0.58 \cdot F_y) \cdot A_g$  and factored rupture resistance  $\phi \cdot V_{nu} = \phi_{vu} \cdot (0.58 F_u) \cdot A_n$  per the FHWA Guidance for Load Rating Evaluation of Gusset Plates in Truss Bridges, LRFR Method. Determine the factored shear resistance  $\phi \cdot V_n$ , which is the smaller of  $\phi \cdot V_{ny}$  and  $\phi \cdot V_{nu}$ . Use  $\phi_{vy} = 0.95$ ,  $\phi_{vu} = 0.80$ , and  $\Omega = 0.74$ .
  - Determine rating factors:
 
$$RF_{inv} = (\phi \cdot V_n - 1.3 \cdot V_{DL}) / 2.17 \cdot V_{HS20LL}$$

$$RF_{op} = (\phi \cdot V_n - 1.3 \cdot V_{DL}) / 1.3 \cdot V_{HS20LL}$$
 If pedestrian live load is present, include entire amount when calculating rating factors:
 
$$RF_{inv} = (\phi \cdot V_n - 1.3 \cdot V_{DL} - 2.17 \cdot V_{ped LL}) / 2.17 \cdot V_{HS20LL}$$

$$RF_{op} = (\phi \cdot V_n - 1.3 \cdot V_{DL} - 1.3 \cdot V_{ped LL}) / 1.3 \cdot V_{HS20LL}$$
  - Cut a section through the gusset plate at a line normal to the chord that passes through the centerline of the joint and repeat steps above.
- 5) Check block shear in gusset plates.
- Consider all truss joints with tension members. Check block shear considering fracture line for each tension member connection. Use engineering judgement to eliminate checking of similar member connections.
  - Determine nominal and factored resistance using LRFD 6.13.4.
  - Determine rating factors:
 
$$RF_{inv} = (R_r - 1.3 \cdot P_{DL}) / 2.17 \cdot P_{HS20LL}$$

$$RF_{op} = (R_r - 1.3 \cdot P_{DL}) / 1.3 \cdot P_{HS20LL}$$
 If pedestrian live load is present, include entire amount when calculating rating factors:
 
$$RF_{inv} = (R_r - 1.3 \cdot P_{DL} - 2.17 \cdot P_{ped LL}) / 2.17 \cdot P_{HS20LL}$$

$$RF_{op} = (R_r - 1.3 \cdot P_{DL} - 1.3 \cdot P_{ped LL}) / 1.3 \cdot P_{HS20LL}$$

- 6) Check edge buckling of gusset plates.
- Consider all truss joints with compression members. Use engineering judgement to eliminate checking of similar member connections.
  - Check Guide Specs for Strength Design of Truss Bridges Article 1.11 requirement for length of unsupported edge:  

$$b/t \leq 11,000 / \sqrt{F_y}$$
  - If requirement is not met, the edge may need stiffening with an angle.
- 7) Check gusset plate buckling at end of diagonals.
- Consider all truss joints with compression diagonals. Use engineering judgement to eliminate checking of similar member connections.
  - Determine "Whitmore effective width" ( $w_{wh}$ ) for diagonal member.
  - Determine DL and LL compression on a 1 inch wide portion of one gusset plate equal to  $P_{DL} / 2 \cdot w_{wh}$  and  $P_{HS20LL} / 2 \cdot w_{wh}$ .
  - Determine unbraced length of gusset plate in compression beyond the end of diagonal equal to the distance along the centerline of the diagonal from the center of the last row of rivets in the diagonal to the centerline of the closest rivet line in the horizontal chord.
  - Determine buckling capacity for a 1 inch wide column with unbraced length as determined above per Std Specs 10.54.1.1. Use  $K = 1.00$ .
  - Determine rating factors:  

$$RF_{inv} = (\phi P_u - 1.3 \cdot p_{DL}) / 2.17 \cdot p_{HS20LL}$$

$$RF_{op} = (\phi P_u - 1.3 \cdot p_{DL}) / 1.3 \cdot p_{HS20LL}$$
 If pedestrian live load is present, include entire amount when calculating rating factors:  

$$RF_{inv} = (\phi P_u - 1.3 \cdot p_{DL} - 2.17 \cdot p_{ped LL}) / 2.17 \cdot p_{HS20LL}$$

$$RF_{op} = (\phi P_u - 1.3 \cdot p_{DL} - 1.3 \cdot p_{ped LL}) / 1.3 \cdot p_{HS20LL}$$
  - Repeat steps above, but use a  $K = 0.75$ . The range of results for both values of  $K$  will be considered upon completion of the review.
- 8) Check bearing and shear of rivets.
- Consider all truss joints. Use engineering judgement to eliminate checking of similar member connections.
  - Determine number of rivets in single shear and in double shear based on the number and size of plates in the joint.
  - Determine load per rivet due to maximum tension and compression loads for DL and LL.
  - Determine the thickness of plates in bearing for the rivets in single shear and double shear.
  - Determine factored bearing and shear capacity using Std Specs 10.56.1.3.
  - Determine rating factors:  

$$RF_{inv} = (\phi \cdot R - 1.3 \cdot p_{DL}) / 2.17 \cdot p_{HS20LL}$$

$$RF_{op} = (\phi \cdot R - 1.3 \cdot p_{DL}) / 1.3 \cdot p_{HS20LL}$$

If pedestrian live load is present, include entire amount when calculating rating factors:

$$RF_{inv} = (\phi \cdot R - 1.3 \cdot p_{DL} - 2.17 \cdot p_{ped LL}) / 2.17 \cdot p_{HS20LL}$$

$$RF_{op} = (\phi \cdot R - 1.3 \cdot p_{DL} - 1.3 \cdot p_{ped LL}) / 1.3 \cdot p_{HS20LL}$$

- 9) Checking fatigue stress in gusset plates due to riveted connections is deferred in accordance with Appendix F of the *Bridge Preservation, Improvement and Replacement Guidelines for Fiscal Year 2006 through 2008*. A fatigue analysis will be done when fatigue cracks are found or major work is planned for the bridge.
  
- 10) At the completion of each bridge review, provide the following documentation in the form of a printed report and electronic files:
  - a. This design review procedure document.
  - b. Copies of original plan sheets and shop drawings for the truss joints reviewed.
  - c. Copy of the latest inspection report.
  - d. A copy of the truss member load table sheet from the original plan (if available).
  - e. Sketches of the truss joints analyzed in flexure and shear showing the section cuts and dimensions used in the analysis.
  - f. The calculations for the truss joints reviewed, including the date and name of the engineers who did the review and check.
  - g. A summary table that reports all truss joints with an inventory and operating rating.

## Attachment 1

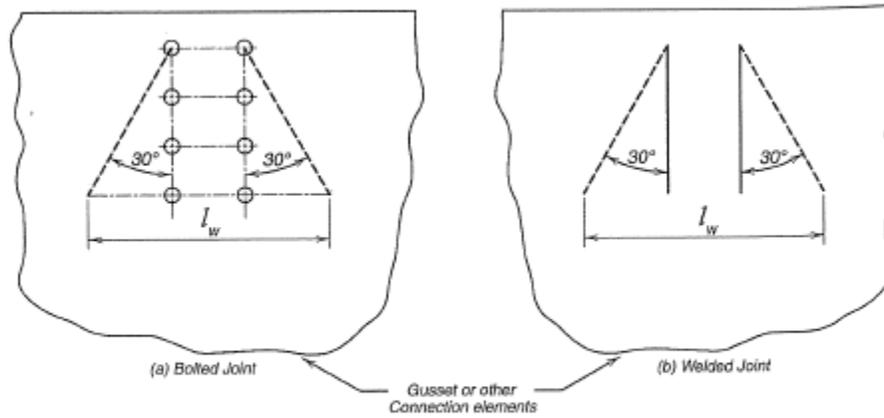


Figure 9-1. Illustration of the width of the Whitmore section.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.

## ***Mn/DOT Interpretation of Truss Bridge Gusset Plate Review Results***

For a given truss, the results of the Mn/DOT Truss Bridge Gusset Plate Design Review include a spreadsheet that calculates inventory and operating rating factors ( $RF_{inv}$  and  $RF_{op}$ ) of the gusset plates and a rating factor summary table for each joint in the truss. Ratings are included for the following scenarios:

- For rivet/bolt shear and bearing check, report ratings for capacity reduction factors  $CRF = 1.00$  and  $CRF = 0.95$ , both with 0% section loss
- For all other checks, report ratings for capacity reduction factors  $CRF = 0.95$  and  $CRF = 0.85$ , both with 0% section loss

After the spreadsheet is complete, a field inspection of the gusset plates must be scheduled to look for corrosion, section loss, missing rivets, or other distress in the plates. All joints must receive a visual inspection. Of particular concern are truss joints located in the salt spray zone, below deck joints, and below deck drains with a calculated  $RF_{op}$  less than or close to 1.00 for a  $CRF = 0.85$ . Check these joints thoroughly to verify that corrosion does not exceed 15%. Other critical joints include those located outside of the salt spray zone (such as joints in the top chord of a high truss) with  $RF_{op}$  less than or close to 1.00 for a  $CRF = 0.95$ . These joints should be field inspected to verify that corrosion does not exceed 5%. For the critical joints where corrosion loss measurements are taken during the inspection, revise the spreadsheet such that  $CRF = 1.00$  and input the actual percentage of section loss.

The long term goal for all truss bridge gusset plates is an adequate rating factor based on analysis using the Mn/DOT Truss Bridge Gusset Plate Design Review Procedure (Mn/DOT Procedure). If this cannot be achieved, the short term goal is an adequate rating factor based on refined analysis until strengthening can be done (within two years). If neither goal is achieved, critical joints must be strengthened immediately. Therefore, the completed rating factor summary table results are to be interpreted in light of the following:

- 1) Joints with an operating rating factor  $RF_{op} \geq 1.00$  for HS20 determined using the Mn/DOT Procedure that meet the edge buckling stiffness requirements of AASHTO LRFD Article 6.14.2.8 are considered adequate. However, all joints with an  $RF_{op} < 1.30$  must also be evaluated for load posting.
- 2) For joints that do not meet the edge buckling stiffness requirements, if the HS20 operating rating factor  $RF_{op} \geq 1.50$  with  $K = 1.00$  for the interior plate buckling check, the joint is considered adequate. If  $RF_{op} < 1.50$ , a refined analysis based on the Dowswell paper and Salmon & Johnson (see below) must be completed. If the joint is shown to be adequate by the refined analysis, the bridge should be scheduled within two years for strengthening of the joint by adding edge stiffeners.

- 3) For joints that meet the edge buckling stiffness requirements, if the HS20 operating rating factor  $RF_{op} \geq 1.00$  with  $K = 0.75$  for the interior plate buckling check, the joint is considered adequate. If  $RF_{op} < 1.00$ , a refined analysis based on the Dowswell paper and Salmon & Johnson (see below) must be completed. If the joint is shown to be adequate using refined analysis, the bridge should be scheduled within two years for strengthening of the joint by adding angles to the compression area of the gusset plate. If it is decided not to strengthen the joint, load posting of the bridge is recommended based on the Mn/DOT Procedure rating.
- 4) For joints with an HS20 operating rating factor  $RF_{op} < 1.00$  for shear governed by the gross section yield criterion, a refined analysis based on the paper by Drucker (see below) must be completed. If the joint is shown to be adequate using refined analysis, the bridge should be scheduled within two years for strengthening of the joint. For joints with  $RF_{op}$  values between 1.00 and 1.10 by the Mn/DOT Procedure, inspect the joint and perform any needed maintenance to guard against further deterioration. If it is decided not to strengthen the joint, load posting of the bridge is recommended based on the Mn/DOT Procedure rating.
- 5) For joints with an HS20 operating rating factor  $RF_{op} < 1.00$  for the rivet bearing/shear check, reanalyze considering the rivet diameter to be equal to the size of the hole. If the joint is shown to be adequate using the hole diameter, the bridge should be scheduled within two years for strengthening of the joint.
- 6) For bridges subject to pedestrian loads that do not result in an acceptable operating rating factor based on the criteria above, consideration should be given to reducing the pedestrian load for calculation of the operating rating factor.

The refined analysis procedures for buckling and shear evaluation are given below. Refined analysis examples are available from the Bridge Office.

Based on the guidance given above, provide a summary report of the final HS20 inventory and operating rating for each truss joint. This will become a page of the final rating form package. Other forms required for the rating form package can be found at:

<http://www.dot.state.mn.us/bridge/DocumentsFormsLinks/discDOCS.html>

Refined Plate Buckling Check Per Dowswell Paper and Salmon & Johnson Book

This method is based on:

Bo Dowswell, *Effective Length Factors for Gusset Plate Buckling*, AISC Engineering Journal, 2<sup>nd</sup> Quarter, 2006

Salmon & Johnson, *Steel Structures: Design and Behavior*, 3<sup>rd</sup> Edition, Harper Collins Publishers Inc., 1990

Procedure:

- 1) Check whether section is compact (whether section can reach yield stress before sidesway buckling occurs) using Dowswell method.

The section is compact for Dowswell method if:

$$t_{gp} \geq t_{\beta}$$

where  $t_{gp}$  = gusset plate thickness

$$t_{\beta} = 1.5 \cdot \sqrt{[F_y \cdot c^3 / (E \cdot l_1)]}$$

$F_y$  = yield stress

$c$  = minimum clear distance between last line of rivets in diagonal and rivet line in chord/vertical

$E$  = elastic modulus

$l_1$  = clear distance between last line of rivets in diagonal and rivet line in chord/vertical measured along diagonal centerline

- 2) Check whether section is compact (whether section can reach yield stress before sidesway buckling occurs) using Salmon & Johnson method.

Plates under uniform compression are governed by:

$$F_{cr} = k \cdot \left\{ \pi^2 \cdot E / [12 \cdot (1 - \mu^2) \cdot (b/t)^2] \right\}$$

where  $F_{cr}$  = elastic critical buckling stress for plates, ksi

$k$  = buckling coefficient

$E$  = elastic modulus, ksi

$\mu$  = Poisson's ratio

$b$  = width of rectangular plate

$t$  = thickness of plate

Determine  $F_{cr}$  for gusset plate assuming an equivalent rectangular plate with the following characteristics:

$$E = 29000 \text{ ksi}$$

$$\mu = 0.3$$

b = width of equivalent rectangular plate  
a = length of equivalent rectangular plate  
k = value from attached Figure 6.15.2 from Salmon & Johnson for fixed-fixed supports along unloaded edges and loaded edges fixed (dashed Curve A)

The section is compact for the Salmon & Johnson method if  $F_{cr} > F_y$

- 3) Determine the capacity of the section in compression.

The section is considered compact for buckling only if it satisfies both of the compactness criteria found in 1) and 2).

If section is compact:

$$\phi P_u = \phi \cdot F_y \cdot W_{wh} \cdot t$$

where  $\phi = 0.9$

$F_y$  = yield stress

$W_{wh}$  = Whitmore effective width

t = thickness of plate

If section is noncompact:

Determine  $\phi P_{uD}$  based on AASHTO Std. Specs. 10.54.1.1, using  $\phi = 1.0$ ,  $K = 1.0$ , and a buckling length  $L_c$  equal to the average  $L_1$ ,  $L_2$ , and  $L_3$  per Dowswell paper.

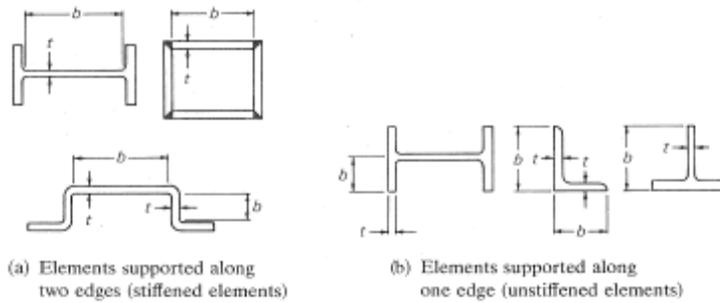
Determine  $\phi P_{uSJ}$  based on AASHTO Std. Specs. 10.54.1.1, using  $F_{cr}$  (if  $F_{cr} > F_y$ , take  $F_{cr} = F_y$ ) calculated per Salmon & Johnson method.

Take  $\phi P_u$  as equal to the smallest of the values  $\phi P_{uD}$  and  $\phi P_{uSJ}$  calculated by the two above methods.

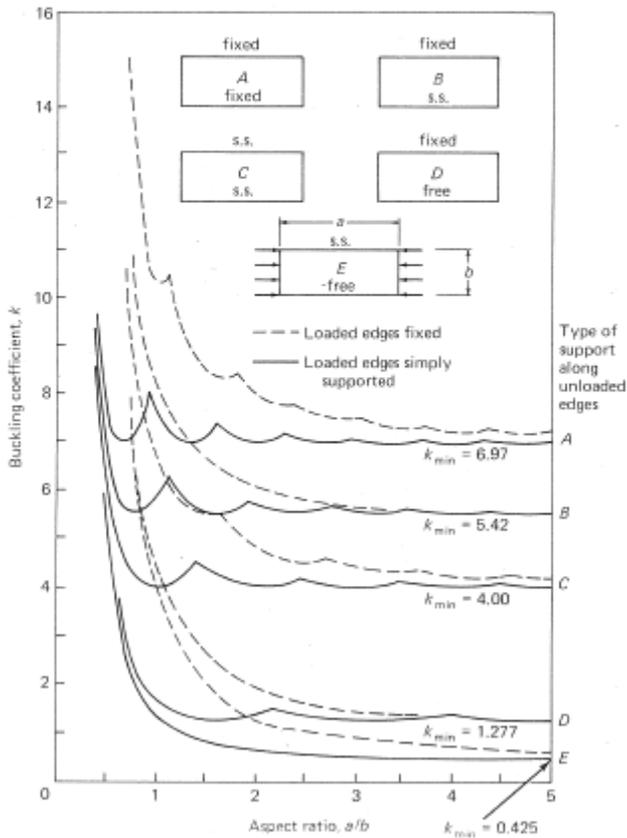
- 4) Determine HS20 operating rating factor  $RF_{op}$  based on  $\phi P_u$  calculated previously in 3).

Figure 6.15.2 taken from:

Salmon & Johnson, *Steel Structures: Design and Behavior*,  
 3<sup>rd</sup> Edition, Harper Collins Publishers Inc., 1990



**Fig. 6.15.1** Stiffened and unstiffened compression elements.



**Fig. 6.15.2** Elastic buckling coefficients for compression in flat rectangular plates. (Adapted from Gerard and Becker [43])

Refined Shear Check Per Drucker Paper

This method is based on:

D. C. Drucker, *The Effect of Shear on the Plastic Bending of Beams*, National Applied Mechanics Division Conference, Urbana, IL, 1956, ASME

The paper considers the effects of normal stress acting in conjunction with shear at the critical section of a stable plate. Drucker recommends the following interaction equation:

$$M_u / M_0 \leq 0.98 \cdot [1 - (V_u / V_0)^4]$$

where  $M_u$  = factored applied moment

$M_0$  = plastic moment capacity

$V_u$  = factored applied shear

$V_0$  = plastic shear capacity =  $0.58 \cdot F_y \cdot A$

Solve for  $V_u$  and add a resistance factor  $\phi$ :

$$V_u \leq \phi \cdot [1 - (M_u / 0.98 \cdot M_0)]^{1/4} \cdot (0.58 \cdot F_y \cdot A)$$

or

$$V_u \leq \phi \cdot \Omega \cdot (0.58 \cdot F_y \cdot A) \quad (\text{similar to equation found in FHWA guidance})$$

where  $\Omega = [1 - (M_u / 0.98 \cdot M_0)]^{1/4}$

Note that for  $M_u = M_y$ ,  $V_u = 0.75V_0$  which is  $\approx$  AASHTO value of  $0.74V_0$

Procedure:

1) Determine factored applied shear  $V_u$  and moment  $M_u$ .

2) Determine plastic moment capacity  $M_0$  :

$$M_0 = 1.5 \cdot M_y = 1.5 F_y \cdot S$$

3) Determine shear/moment interaction reduction factor  $\Omega$  :

$$\Omega = [1 - (M_u / 0.98 \cdot M_0)]^{1/4}$$

4) Determine factored shear capacity  $\phi \cdot V_n$ , which is the smaller of:

$$\phi \cdot V_{ny} = \phi_{vy} \cdot \Omega \cdot (0.58 \cdot F_y \cdot A_g)$$

$$\phi \cdot V_{nu} = \phi_{vu} \cdot (0.58 \cdot F_u \cdot A_n)$$

5) Determine HS20 operating rating factor  $RF_{op}$  based on  $\phi \cdot V_n$  calculated previously in 4).



**Minnesota Department of Transportation**

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# Memo

**TO:** Bridge Design Engineers  
Construction Managers Group (CMG)  
Resident Engineers

**FROM:** Kevin Western *Kevin Western*  
State Bridge Design Engineer

**DATE:** April 12, 2012

**MEMO TO DESIGNERS (2012-01):** Discontinued Usage of Plain Elastomeric Bearing Pads and Substitution with Cotton-Duck Bearing Pads

In light of some recent excessive deformations of plain elastomeric bearing pads (PEP), the use of PEP will be limited. At this time, the precise cause of the performance issue has not been determined. Further research is being completed by the AASHTO Bearing Committee to isolate the source of the issue. When the research is complete, final guidance on plain elastomeric bearing pads will be issued. The guidance provided in this memo should be used in the interim.

Plain elastomeric bearing pads are used at fixed bearing assemblies and at integral abutments. There will be no change to the use of PEP at integral abutments. Because the bearing pads are confined by the concrete and the polystyrene, the amount of deformation is limited and is not a concern.

For all fixed curved plate bearing assemblies for both steel and prestressed beams, replace the plain elastomeric bearing pad with a cotton-duck bearing pad (CDP) of the same size as required for a PEP. CDP are preformed pads that are produced in large sheets and cut to size for specific bridge applications. CDP are reinforced with closely spaced layers of cotton-duck and typically display high compressive stiffness and strength, obtained by the use of very thin elastomeric layers. These pads are often used on railroad structures due to their high compressive strengths.

Cotton-duck pads must be manufactured and tested under compression in accordance with Military Specification MIL-C-882E, except where superseded by the current AASHTO LRFD Bridge Design Specifications Article 14.7.6.2 or by this memo. The minimum low-temperature grade of elastomer for cotton-duck pads is Grade 3.

This change to the bearing pads is effective for all bridges where the bearings have not yet been installed. For bridges still in the design phase that utilize standard detail B310 or B354, include revised special provision SB2012-3741 that is attached to this memo. For projects that have been submitted to CO, but not yet let, an addendum will be necessary. For projects that have already been let, please coordinate and assist the districts to obtain CDP through the



supplemental agreement process. As a minimum, these changes will be in effect for the entire 2012 construction season. This guidance will remain in place until final guidance supersedes this memo.

If you have any questions contact myself or Arielle Ehrlich at (651) 366-4515.

Thank you.

cc: Dave Conkel/Local Consultants  
Dave Dufresne/PACAL Industries, LLC  
Bob Timpane/Construction Materials Inc.  
Craig Kirks/D.S. Brown  
Jim Kochsiek (MS 645)  
Steve Grover (MS 645)



**Use on all jobs requiring Elastomeric Bearing Pads.**

**(1) Use where standard details B310 and B354 are used.**

**Created: 3/22/2002**

**Revised: 4/12/2012 (7)**

SB- (3741) ELASTOMERIC BEARING PADS

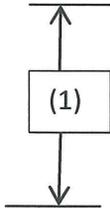
Apply the provisions of 3741 except as modified below:

Replace the first sentence in 3741.2A with the following:

Provide elastomer for bearing pads meeting the requirements of AASHTO M 251 with durometer hardness of  $60 \pm 5$  on the Shore "A" scale. Provide elastomer compounds classified as low-temperature Grade 4 meeting the grade requirements of AASHTO *LRFD Bridge Design Specifications*, Table 14.7.5.2-1, "Low temperature Zones and Minimum Grade of Elastomer."

Delete all of 3741.2B1 except for the last paragraph.

Utilize cotton-duck bearing pads (CDP) where standard details B310 and B354 are used. Test and manufacture CDP in accordance with Military Specification MIL-C-882E, except where superseded by the current AASHTO LRFD Bridge Design Specifications Article 14.7.6.2 or by Memo to Designers (2012-01). For CDP, provide elastomer compounds classified as low-temperature Grade 3.







**Minnesota Department of Transportation**

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**Memo**

**TO:** Bridge Design Engineers  
Construction Managers Group (CMG)  
Resident Engineers

**FROM:** Arielle Ehrlich *Arielle Ehrlich*  
State Bridge Design Engineer

**DATE:** November 21, 2012

**MEMO TO DESIGNERS (2012-02):** Transition to New MnDOT Pile Formula 2012 (MPF12)

Based on the results of a recent MnDOT research project, MnDOT will discontinue the use of the current MnDOT Nominal Resistance Pile Driving Formula (known as the LRFD formula) for construction control and replace it with a new pile driving formula designated the MnDOT Pile Formula 2012 (MPF12). The MPF12, which was calibrated for site conditions typically encountered in Minnesota, more reliably predicts the nominal resistance of driven piles using LRFD methodology.

This change in formula will not affect the design of bridges, but will require a change in the standard plan notes. The Required Nominal Pile Bearing Resistance Tables from Appendix 2-H Article F, page 2-104, of the MnDOT LRFD Bridge Design Manual are hereby replaced with the attached tables.

For the MPF12, note that CIP piles and H-piles have different resistance factors ( $\phi_{dyn}$ ). As a result, there are now four tables presented for use depending on the type of pile used.

A revised special provision will be released shortly that covers the use of the MPF12. The special provision will include limitations on MPF12 usage, such as blow count range and pile hammer qualifications.

In order to educate field personnel, the Bridge Construction Unit will organize a number of training sessions statewide to provide information regarding the transition to the MPF12. These sessions will be beneficial for personnel from MnDOT Districts, MnDOT State Aid, consultant inspectors, and geotechnical consultants. The tools used by inspectors to calculate driven pile capacities and pile forms will also be revised to reflect the new formula. Look for more information in the coming months.

This new formula should be utilized for all bridge projects with letting dates after February 28, 2013.

If you have any questions contact me or Dave Dahlberg at (651) 366-4491.

Thank you.

cc: Dave Conkel/Local Consultants  
Colleen Harer/Design Consultants  
Gary Person/Foundations (MS 645)

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Current standard plan notes, Appendix 2-H, Article F

<b>_____ ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE <math>R_n</math> - Tons/Pile</b>		
FIELD CONTROL METHOD	$\Phi_{dyn}$	* $R_n$
Mn/DOT Nominal Resistance Formula	0.40	
PDA	0.65	

\*  $R_n = (\text{Factored Design Load}) / \Phi_{dyn}$

<b>PIER _____ REQUIRED NOMINAL PILE BEARING RESISTANCE <math>R_n</math> - Tons/Pile</b>		
FIELD CONTROL METHOD	$\Phi_{dyn}$	* $R_n$
Mn/DOT Nominal Resistance Formula	0.40	
PDA	0.65	

\*  $R_n = (\text{Factored Design Load}) / \Phi_{dyn}$

Revised standard plan notes, Appendix 2-H, Article F

H-Piles

<b>_____ ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE FOR H-PILES <math>R_n</math> - Tons/Pile</b>		
FIELD CONTROL METHOD	$\Phi_{dyn}$	* $R_n$
MnDOT Pile Formula 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.60	
PDA	0.65	

\*  $R_n = (\text{Factored Design Load}) / \Phi_{dyn}$

<b>PIER _____ REQUIRED NOMINAL PILE BEARING RESISTANCE FOR H-PILES <math>R_n</math> - Tons/Pile</b>		
FIELD CONTROL METHOD	$\Phi_{dyn}$	* $R_n$
MnDOT Pile Formula 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.60	
PDA	0.65	

\*  $R_n = (\text{Factored Design Load}) / \Phi_{dyn}$

CIP Piles

<b>_____ ABUTMENT REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES <math>R_n</math> - Tons/Pile</b>		
FIELD CONTROL METHOD	$\Phi_{dyn}$	* $R_n$
MnDOT Pile Formula 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	
PDA	0.65	

\*  $R_n = (\text{Factored Design Load}) / \Phi_{dyn}$

<b>PIER _____ REQUIRED NOMINAL PILE BEARING RESISTANCE FOR CIP PILES <math>R_n</math> - Tons/Pile</b>		
FIELD CONTROL METHOD	$\Phi_{dyn}$	* $R_n$
MnDOT Pile Formula 2012 (MPF12) $R_n = 20 \sqrt{\frac{W \times H}{1000}} \times \log\left(\frac{10}{S}\right)$	0.50	
PDA	0.65	

\*  $R_n = (\text{Factored Design Load}) / \Phi_{dyn}$



# Minnesota Department of Transportation

MS 610  
3485 Hadley Avenue North  
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Office Tel: (651) 366-4506  
Office Fax: (651) 366-4497

## Memo

**TO:** Bridge Design Engineers

**FROM:** Arielle Ehrlich   
State Bridge Design Engineer

**DATE:** April 17, 2013

### **MEMO TO DESIGNERS (2013-01):** Conversion from Metric to U.S. Customary Rebar Designations

Recently, the Concrete Reinforcing Steel Institute, a consortium of rebar producers and fabricators, has announced that its members have begun the process of converting from metric rebar designations back to U.S. customary designations. U.S. customary bar designations are indicated by a one or two digit number, equal to the nominal bar diameter in eighths of an inch (i.e., a No. 4 bar has a diameter of 4/8, or 1/2 inch).

For all projects scheduled for letting on or after July 1, 2013, include bar marks in U.S. customary bar sizes. MnDOT has issued newly updated roadway and bridge standard plans and plates in U.S. customary bar sizes. Additionally, the CADD cell library which contains the Standard Plan Notes has been updated for U.S. customary bar sizes. All updated files are available on the web and internal server sites at the same location as the previous standards.

Effective immediately, modify Table 2472-2 in the 2005 Edition of the MnDOT Standard Specifications for Construction using the special provision found in MnDOT Technical Memorandum 13-06-B-03.

The entirety of this manual will not be republished immediately, but all future updates will use U.S. customary bar marks.

For more information, see MnDOT Technical Memorandum 13-06-B-03.

If you have any questions contact myself or Dave Dahlberg at (651) 366-4491.

Thank you.

cc: Nancy Daubenberger  
Paul Rowekamp  
Dave Dahlberg  
Dave Conkel/Local Consultants  
Colleen Harer/Design Consultants







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## Memo

**TO:** Bridge Design Engineers

**FROM:** Arielle Ehrlich   
State Bridge Design Engineer

**DATE:** August 6, 2014

### MEMO TO DESIGNERS (2014-01): AASHTO LRFD Article 5.7.3.4 Concrete Crack Control Check

With the "JULY 2014" revision to the *LRFD Bridge Design Manual (BDM)*, some clarification is needed on the crack control check until it can be added to Section 11 of the *BDM*.

Use the following guidance when checking crack control:

- Retaining walls and substructures (excluding pier caps)
  - Use Class 1 exposure condition ( $\gamma_e = 1.00$ ).
  - Use actual clear cover to flexural bars when calculating  $d_c$ .
- Pier caps
  - Use Class 2 exposure condition ( $\gamma_e = 0.75$ ).
  - Use actual clear cover to flexural bars when calculating  $d_c$ .
- Slabs and decks
  - Use Class 2 exposure condition ( $\gamma_e = 0.75$ ).
  - Although the actual clear cover may exceed 2 inches for the slab/deck top bars, calculate  $d_c$  using a maximum clear concrete cover equal to 2 inches.
- Footings
  - Crack control check is not required.

If you have any questions, contact myself or Dave Dahlberg at (651) 366-4491.

Thank you.

cc: Nancy Daubenberger  
Dave Dahlberg  
Dave Conkel/Local Consultants  
Colleen Harer/Design Consultants







**Minnesota Department of Transportation**

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**Memo**

**TO:** Bridge Design Engineers  
**FROM:** Arielle Ehrlich *Arielle Ehrlich*  
State Bridge Design Engineer  
**DATE:** December 23, 2014

**MEMO TO DESIGNERS (2014-02):** Inclusion of Informational Quantities in Bridge Plans

Following recent policy changes for grading plans and at the request of the Office of Construction and Innovative Contracting, bridge plans will no longer include informational quantities for contractor convenience. Examples of these items are summary quantities for conduit systems, summary quantities for drainage systems, cubic yard quantities for concrete items paid for by the square foot or linear foot, and miscellaneous incidental items, such as polystyrene, waterproofing membrane, and cork. All new plans or plans at the beginning stages of final design should follow these new guidelines; plans that are complete or nearly complete need not be changed.

Although these quantities will no longer be incorporated into bridge plans, the quantities will still need to be computed and submitted to the Bridge Estimating Unit. As an aid, sample forms are available at <http://www.dot.state.mn.us/bridge/design.html>.

If you have any questions, please contact Jeff Southward ([jeff.southward@state.mn.us](mailto:jeff.southward@state.mn.us) or (651) 366-4452) or me.

- cc: Nancy Daubenberger
- Dave Dahlberg
- Jeff Southward
- Bob Schabacker
- Dave Conkel/Local Consultants
- Colleen Harer/Design Consultants







# Minnesota Department of Transportation

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## Memo

**TO:** Bridge Design Engineers

**FROM:** Arielle Ehrlich   
State Bridge Design Engineer

**DATE:** August 10, 2015

### MEMO TO DESIGNERS (2015-01): Concrete Mix Design Designations

With the publishing of the 2016 Edition of the MnDOT *Standard Specifications for Construction* (2016 MnDOT Specs.), MnDOT is switching to Contractor mix designs for concrete. One of the results of this change is new concrete mix designations. Beginning with the October 23, 2015 letting, all projects will utilize the 2016 MnDOT Specs.

In order to decrease the work involved in making the transition, abbreviated changes are allowed for bridge plans that are let in October, November, or December of 2015, in accordance with the following:

- In the “CONSTRUCTION NOTES” placed on the “GENERAL PLAN AND ELEVATION” sheet, add the following note:  
    CONVERT OLD CONCRETE MIX DESIGN DESIGNATIONS SHOWN IN PLANS  
    TO 2016 CONCRETE MIXES USING TABLES 2461-6 AND 2462-6 FOUND IN  
    THE PROJECT SPECIAL PROVISIONS, DIVISION S.
- In the “Schedule of Quantities” table placed near the front of the plan, replace each 2014 MnDOT Spec. pay item that contains an old concrete mix designation with the 2016 MnDOT Spec. pay item that contains the new concrete mix designation.
- For each pay item that has been replaced as stated above, include a circled note below the table stating that the new pay item replaces all references to the old pay item found throughout the bridge plan.
  - Examples:
    - ① PAY ITEM “STRUCTURAL CONCRETE (1G52)” REPLACES ALL REFERENCES TO “STRUCTURAL CONCRETE (1A43)” THROUGHOUT THIS PLAN.
    - ② PAY ITEM “TYPE F (TL-4) BARRIER CONCRETE (3S52)” REPLACES ALL REFERENCES TO “TYPE F (TL-4) RAILING CONCRETE (3Y46)” THROUGHOUT THIS PLAN.



② PAY ITEM "TYPE F (TL-4) BARRIER CONCRETE (3S52)" REPLACES ALL REFERENCES TO "TYPE F (TL-4) RAILING CONCRETE (3Y46)" THROUGHOUT THIS PLAN.

For all bridge plans in the January 2016 letting and thereafter, provide the new concrete mix designations throughout the plan and do not include plan notes for conversion.

The Trns\*port pay item list has been updated and can be found at:

<http://bidlet.dot.state.mn.us/english2016.aspx>

The Division "SB" bridge special provisions have been updated and can be found at:

<http://www.dot.state.mn.us/bridge/construction.html>

The LRFD Bridge Design Manual (BDM) Table 5.1.1.1, "Design Concrete Mix Summary", has been updated to the new concrete mix designations. However, other locations where a concrete mix is listed within the BDM have not been updated, but will be when the section they are in is revised. A conversion table for bridge elements is attached as a reference.

If you have any questions, please contact Dave Dahlberg ([dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or (651) 366-4491) or me.

cc: Bev Farraher  
Dave Dahlberg  
Dave Conkel/Local Consultants  
Colleen Harer/Design Consultants



**Concrete Mix Design Conversion Table  
for Bridge and Structural Elements**

Location/Element	Old MnDOT Mix Designation	New MnDOT Mix Designation
Cofferdam seals	1X62	1X62
Cast-in-place concrete piles and spread footing leveling pads	1C62	1P62
Drilled shafts	1X46 1Y46	1X62 3X62
Footings and pile caps	1A43	1G52
Abutment stems, wingwalls, cast-in-place wall stems, pier columns, and pier caps	3Y43	3B52
Integral abutment diaphragms and pier continuity diaphragms	Same mix as used in deck	Same mix as used in deck
Pretensioned superstructures	1W36 3W36 special	1W82 3W82 N/A
Cast-in-place and precast box girders	3JM	3JM
Monolithic decks and slabs	3YHPC-M 3YLCHPC-M 3Y33	3YHPC-M 3YLCHPC-M 3Y42
Decks and slabs that will receive a 2 inch concrete wearing course	3YHPC-S 3YLCHPC-S 3Y36	3YHPC-S 3YLCHPC-S 3Y42
Barriers, end posts, parapets, median barrier, raised medians, and sidewalks	3Y46	3S52
Concrete wearing course	3U17A	3U17A
MSE wall panels, PMBW blocks, and noisewall panels	3Y43	3Y82
Precast box culverts, arches, and 3-sided structures	3W36	3W82







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# Memo

**TO:** Bridge Design Engineers

**FROM:** Arielle Ehrlich *Arielle Ehrlich*  
State Bridge Design Engineer

**DATE:** December 9, 2016

### MEMO TO DESIGNERS (2016-01): Single Slope Barrier (Type S) Bridge Standards

New bridge barrier standards have been developed for use that incorporate a single slope shape on the front face. The single slope shape has been shown to impart less climb and instability to passenger vehicles during a crash. The new MnDOT Type S bridge barriers were developed based on the Texas Type SSTR barrier. The Texas Type SSTR barrier was developed to meet the crash testing requirements found in the AASHTO Manual for Assessing Safety Hardware 2009 (MASH) Test Level 4 (TL-4) and was successfully crash tested to that level. AASHTO has not yet published a table of MASH test level design forces that can be used for deck overhang design or other components of the bridge. Therefore, although the barriers as developed meet the MASH criteria for geometry and are expected to meet the MASH criteria for strength, the new Minnesota Type S barrier standards have been published as meeting the crash testing requirements of NCHRP 350 Test Level 4.

#### Available Type S Bridge Standards

Standards are available for three barrier heights: 36", 42", and 54". Variations are included for:

- bridges with integral, semi-integral, and parapet type abutments
- bridges with and without a wearing course
- bridges with and without a sidewalk
- bridges with split median barriers
- bridges with solid median barriers

#### Detailing Requirements

A single slope TL-5 barrier has not been developed at this time. For bridges where a TL-5 barrier would currently be recommended due to high design speeds, curvature, truck traffic, or other site considerations (see LRFD Bridge Design Manual (BDM) Article 13.2.1), use a TL-4 42" tall Type S barrier or TL-4 54" tall Type S barrier where a glare screen is needed.



For bridges with wingwall orientation parallel to the roadway that have the barrier located on top of the wingwalls, detail the wingwalls to be the same thickness as the barrier (1'-4") for the top 1'-6" and then transition to the full standard thickness. See Figure 1.

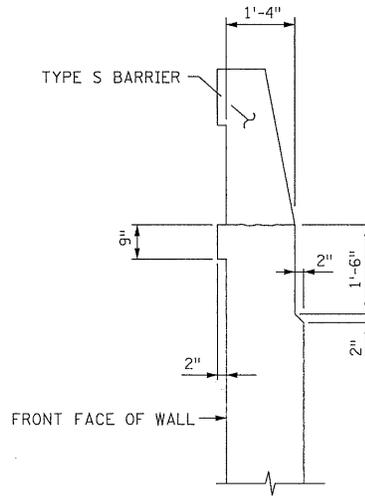


Figure 1

The Midwest Roadside Safety Facility (MwRSF) has provided recommendations for barrier placement on bridges that are dependent on the cross slope. MwRSF recommends limiting the angle between the roadway surface and the vertical axis of the barrier to a maximum of 90 degrees. Use the following guidance to meet these recommendations. Note that this guidance is similar to what was found in the Memo to Designers (2011-02), which is now rescinded:

- For driving surfaces with a normal crown section, detail the barriers as plumb. See Figure 2 below.

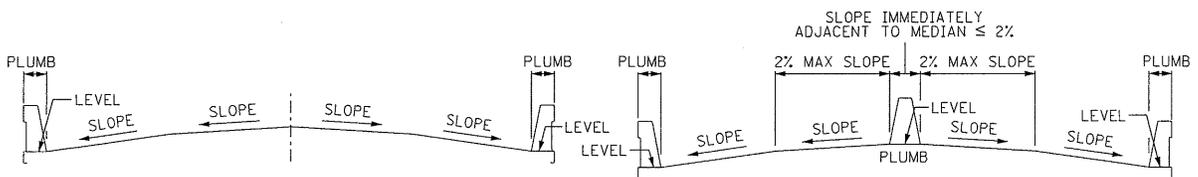


Figure 2

- For driving surfaces with a constant cross slope (superelevated roadway) exceeding 2%, detail the angle between the bridge deck/roadway and the vertical axis of the barrier so it does not exceed 90 degrees. See Figure 3 below.



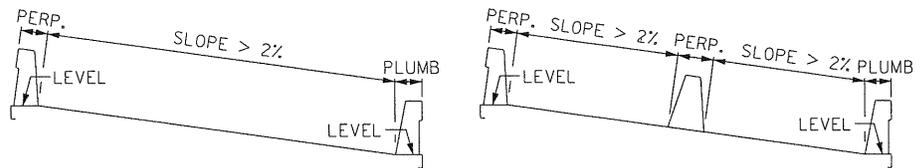


Figure 3

- For bridge decks with a variable or changing cross slope or superelevation, detail the angle between the bridge deck/roadway and the vertical axis of the barrier to transition from plumb to perpendicular (or vice versa) as shown in Figure 4. In this example, the cross section changes from a “normal” cross slope to a superelevated cross slope, so the left barrier transitions from plumb at the “normal” cross slope to perpendicular at 0% slope.

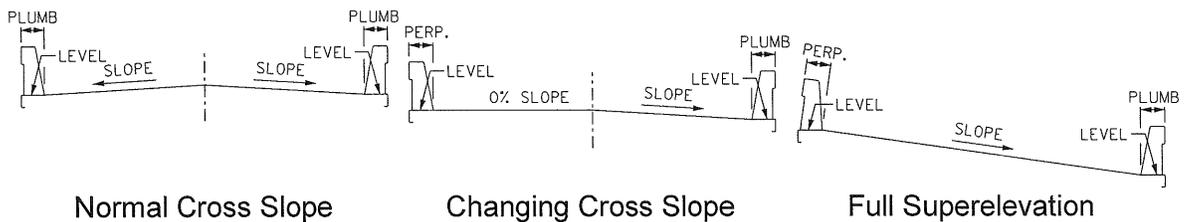


Figure 4

- As indicated by the examples above, the vertical position of the barrier axis varies depending on the adjacent driving surface slope; therefore, it is imperative that the bridge and roadway designer work together to ensure that design plans are coordinated and that the detailing on the bridge plan matches the roadway plan and vice versa. Consideration of the barrier axis must also be taken into account when the barrier is mounted on top of a wall or approach panel.
- Regardless of whether the barrier is mounted plumb or perpendicular to the roadway surface, the portion of the deck immediately under the barrier should remain level as is shown in the sketches above. An exception to this is solid median barriers on superelevated cross-slopes.
- Where the cross slope exceeds 2%, include barrier height dimensions for both the front and back face of barriers located at the top of the slope. In addition, revise the R501E, R502E, and R503E bars to provide a minimum front leg projection of 10 inches.



## Deck Overhang Design

Deck overhang requirements are dependent on the overhang length and overhang location along the bridge.

- For deck overhangs that carry a Type S barrier (measured from centerline of beam to edge of deck) of up to 40% of the beam spacing, the BDM deck reinforcement tables 9.2.1.1 and 9.2.1.2 may be used for the interior overhang regions. For the exterior overhang regions (applies to regions where the longitudinal barrier reinforcement is discontinuous, such as end of bridge joints and expansion joints), the following modifications to the overhang reinforcement are necessary to meet NCHRP 350:
  - For the 36" Type S, provide #5 bars at 5" spacing or  $A_s = 0.74 \text{ in}^2/\text{ft}$  for the top transverse bars over a distance of 8 feet from the joint. Include 180 degree standard hooks on the edge-of-deck ends of these bars. This can be accomplished either by providing hooked overhang bars that splice to the main transverse deck bars or by providing hooked transverse bars that run from edge to edge of the deck. Note that this only applies when the gutter line is located outside the edge of the fascia beam flange. For cases where the gutter line is located inside the edge of the fascia beam flange, provide reinforcement per the BDM deck reinforcement tables 9.2.1.1 and 9.2.1.2 with no modification.
  - For the 42" Type S where the deck consists of a 9" slab without a wearing course, include 180 degree standard hooks on the edge-of-deck ends of the top transverse bars over a distance of 9 feet from the joint. This can be accomplished either by providing hooked overhang bars that splice to the main transverse deck bars or by providing hooked transverse bars that run from edge to edge of the deck. For all other cases, provide reinforcement per the BDM deck reinforcement tables 9.2.1.1 and 9.2.1.2 with no modification. Also, note that for all cases where the gutter line is located inside the edge of the fascia beam flange, no modification is needed and reinforcement per the BDM deck reinforcement tables 9.2.1.1 and 9.2.1.2 can be provided.
  - For the 54" Type S, no modification is needed. Provide reinforcement per the BDM deck reinforcement tables 9.2.1.1 and 9.2.1.2.
  - BDM Figure 9.2.1 specifies an 8" minimum edge-of-deck thickness when the deck includes a wearing course, and a 9" minimum edge-of-deck thickness when there is not a wearing course. Where the "Deck Thickness" column in the BDM deck reinforcement tables 9.2.1.1 and 9.2.1.2 specifies a thickness greater than 9", increase the edge-of-deck thickness by the difference between the specified deck thickness and 9". For example, for a steel beam bridge with a 12'-0" beam spacing, BDM table 9.2.1.2 specifies a 9.75" deck thickness. If the bridge has a wearing course, use an edge-of-deck thickness of 8.75". If the bridge does not have a wearing course, use an edge-of-deck thickness of 9.75". Also note that for bridges with wingwalls oriented parallel to the roadway or that tie into a retaining wall, adjust the wingwall/retaining wall coping height as needed to match the edge-of-deck thickness. Consult



with the Bridge Office Architectural Specialist for guidance.

- For deck overhangs that require a special design, use the following guidance for checking the extreme event limit state.

Design collision loads  $F_{cdes}$  and  $M_{cdes}$  will be needed to complete the deck overhang design. For each barrier height  $H$ , values for  $L_c$ ,  $M_c$ , and  $R_w$  have been determined using the yield line method found in AASHTO LRFD Spec. Article A13.3.1. The TL-4 value of 54 kips for  $F_t$  was adjusted for the difference between the barrier height and height of  $F_t$  application. Then  $4/3 \cdot F_t$  was compared to  $R_w$ , and the smaller value distributed over  $L_c + H$  for end regions and  $L_c + 2H$  for interior regions. Also,  $M_c$  was adjusted when  $4/3 \cdot F_t$  governed. The results are the moments  $M_{cadj}$  and tension forces  $F_{cadj}$  given in Table 1 below.

	36" Type S		42" Type S		54" Type S	
	Exterior	Interior	Exterior	Interior	Exterior	Interior
$M_{cadj}$ (k-ft/ft)	20.5	9.4	18.8	7.8	17.4	6.7
$F_{cadj}$ (k/ft)	7.9	3.7	6.1	2.7	4.1	1.8

Table 1

In order to use these values, translate the moment  $M_{cadj}$  at the top of the deck to a moment  $M_{cdes}$  located at the center of the deck using the following method (refer to Figure 5):

$$e = M_{cadj} / F_{cadj}$$

$$F_{cdes} = F_{cadj}$$

$$M_{cdes} = F_{cdes} \cdot (e + 0.5 \cdot t_{deck})$$



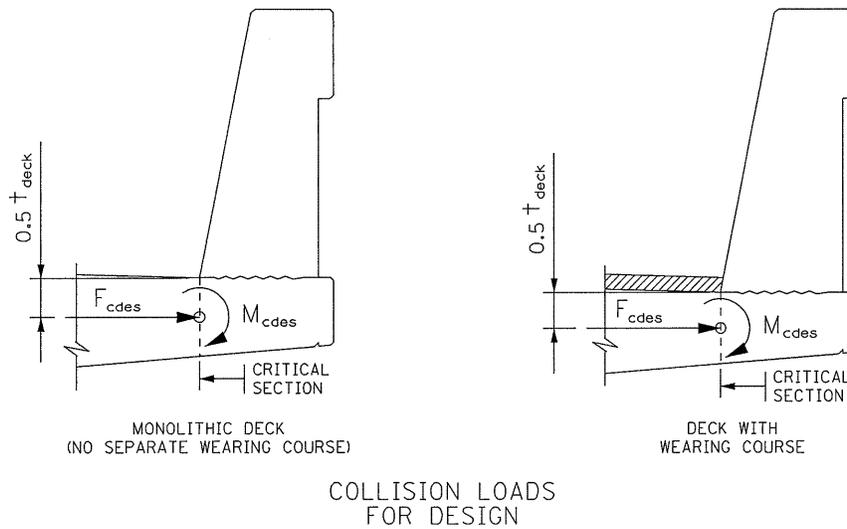
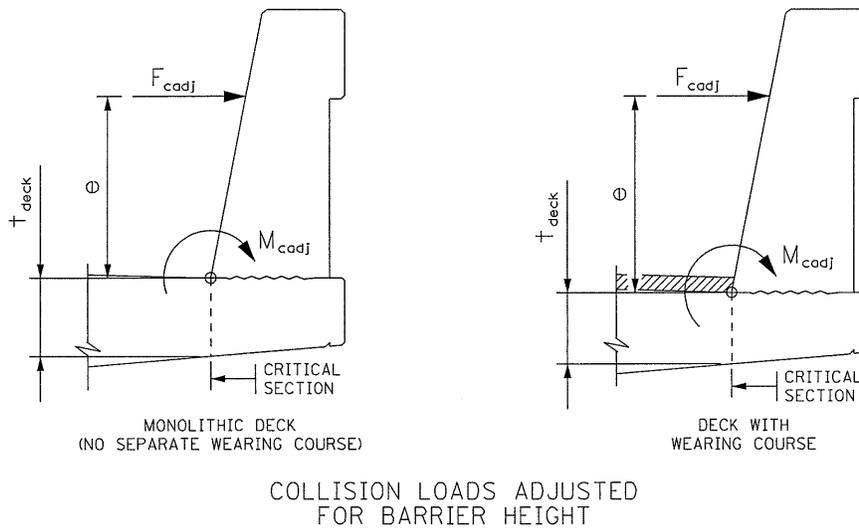


Figure 5

Use the tension force  $F_{cdes}$  and moment  $M_{cdes}$  as the collision loads for the deck overhang design.

Implementation

When to use the Type S barrier will depend on the project.

Use of the single slope bridge barrier is to begin immediately for the following:

- All new preliminary bridge plans for projects on trunk highways where the Type F barrier normally would be used. (The structural tube railing (T-1) and concrete parapet (P-4) will continue to be used where appropriate.)



- New final design bridge plans for projects on trunk highways where the Type S bridge barrier is shown in the preliminary bridge plan.
- Stand-alone new final design bridge plans for projects on trunk highways where Type F bridge barrier is shown in the preliminary bridge plan, but final design has not been started. An exception to this is bridges that connect to retaining walls, which will require coordination with the road designer on the decision regarding barrier type.
- Non-stand-alone new final design bridge plans for projects on trunk highways where Type F bridge barrier is shown in the preliminary bridge plan, but the roadway designer agrees to the use of Type S bridge barrier instead.

The decision on what type of barrier to use for bridge repair projects will be made on a case-by-case basis.

For local road bridge projects, it is recommended that designers immediately begin use of the single slope barrier in accordance with this memo. For questions, consult with the State Aid Bridge Unit.

Beyond the transition period to single slope barriers, Type F barrier should only be used on bridge repair projects.

Note that the Type F temporary portable precast concrete barrier (Standard Plate 8337) has been successfully crash tested to meet MASH TL-3, so it will not be replaced and can continue to be used where applicable.

If you have any questions, please contact Dave Dahlberg ([dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or (651) 366-4491) or me.

cc: Kevin Western  
Dave Dahlberg  
Dave Conkel/Local Bridge Design Consultants  
Colleen Lichtsinn/Bridge Design Consultants





## Memo

To: Bridge Design Engineers

From: Arielle Ehrlich   
State Bridge Design Engineer

Date: Mar. 28, 2017

### MEMO TO DESIGNERS (2017-01): Edge-of-Deck Thickness on Bridges and Wall Coping Height

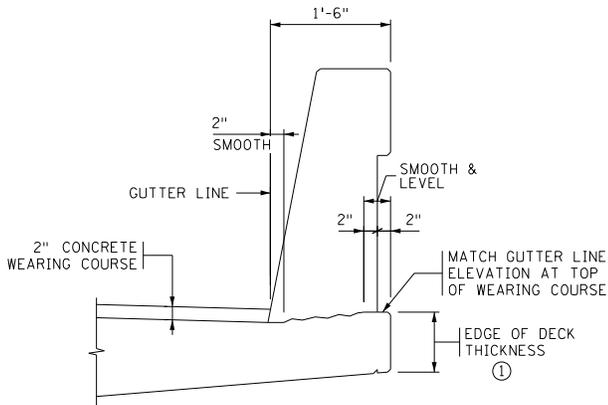
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For deck-on-beam type bridges, MnDOT's *LRFD Bridge Design Manual* (BDM) Figure 9.2.1 currently requires choosing an edge-of-deck thickness based on the type of deck to be constructed. For bridges with monolithic decks, an edge-of-deck thickness equal to 9" is required. When the deck consists of a 7" structural slab plus 2" concrete wearing course, an 8" edge-of-deck thickness is required. The current method for detailing the structural slab plus wearing course causes an abrupt vertical change in the coping line at the end of bridge expansion joints when the wingwalls are oriented parallel to the roadway. In order to simplify the current practice and provide a continuous coping line, the new practice is to require an edge-of-deck thickness that is equal to the deck thickness specified in BDM Tables 9.2.1.1 and 9.2.1.2 for all cases (typically 9", except for wide beam spacings). In the special case where a deck thickness less than 9" is specified (e.g., redecking of a bridge with existing deck thickness equal to 8½"), provide an edge-of-deck thickness equal to 9". For monolithic decks, the result is the same as current practice. For decks that include a concrete wearing course, the new practice will be accomplished by adjusting the top of deck as it passes beneath the barrier so that the edge-of-deck elevation is the same as the gutterline elevation. See Figure 1.

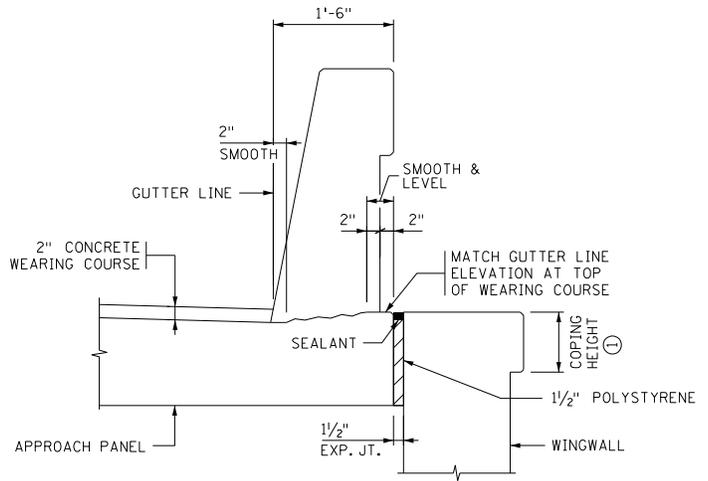
For wingwalls parallel to the roadway, current practice is to provide a coping on the outside face of the wall. For bridges with and without a wearing course, use a coping height that matches the edge-of-deck thickness. This will result in a continuous coping line on and off the superstructure. Figure 1 shows barrier details for the two cases of:

- 1) a barrier constructed on the approach panel
- 2) a barrier constructed on a wingwall

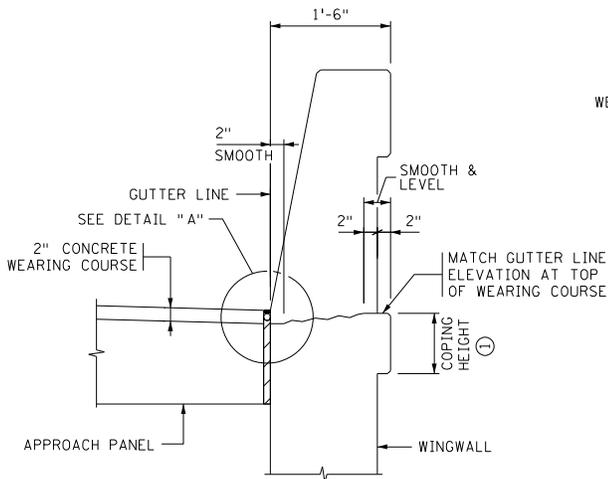
The cast-in-place concrete retaining wall standards currently show an 8" high coping. Revisions to the standards that show a 9" high coping will be published in the near future. Where a wingwall ties into a retaining wall, coordinate with the roadway designer as needed to set the retaining wall coping height to match the edge-of-deck thickness.



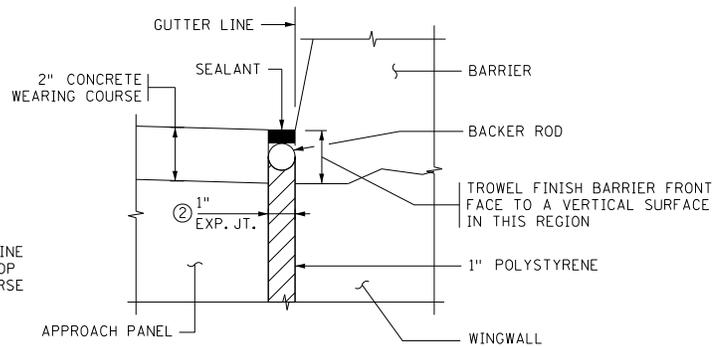
BARRIER SECTION ON DECK



BARRIER SECTION ON APPROACH PANEL



BARRIER SECTION ON WINGWALL



DETAIL A

- ① USE VALUE FOR "DECK THICKNESS" GIVEN IN BDM TABLES 9.2.1.1 AND 9.2.1.2.
- ② POLYSTYRENE, BACKER ROD, AND JOINT SEALANT ARE INCIDENTAL TO APPROACH PANEL CONSTRUCTION.

EDGE OF DECK AND COPING DETAILS FOR BRIDGES WITH A WEARING COURSE

Figure 1

These changes result in the following revisions to previous guidance found in the Memo to Designers (2016-01):

- 1) In the guidance regarding plumb vs. perpendicular barrier placement, the 5<sup>th</sup> bullet as well as Figures 2, 3, and 4 of Memo to Designers (2016-01) called for a level deck surface beneath the barrier. For decks that include a concrete wearing course, the top of deck under the barrier will not be level, but will slope as shown above in Figure 1 of this Memo to Designers. However, the barrier orientation will still remain plumb or perpendicular as defined in the previous memo.
- 2) For the deck overhang design, the net effect of the edge-of-deck change for decks that include a concrete wearing course is to reduce the deck thickness at the gutter line. Therefore, the entire section regarding deck overhang design is superseded by the following:

#### Deck Overhang Design

Deck overhang requirements are dependent on the overhang length and overhang location along the bridge.

- For deck overhangs (measured from centerline of beam to edge of deck) of up to 40% of the beam spacing that carry a Type S barrier, the BDM deck reinforcement Tables 9.2.1.1 and 9.2.1.2 may be used for the interior overhang regions. For the exterior overhang regions (applies to regions where the longitudinal barrier reinforcement is discontinuous, such as end of bridge joints and expansion joints), the following modifications to the overhang reinforcement are necessary to meet NCHRP 350 Test Level 4 (TL-4). Note that the modifications only apply when the gutter line is located outside the edge of the fascia beam flange. For cases where the gutter line is located inside the edge of the fascia beam flange, provide reinforcement per the BDM deck reinforcement Tables 9.2.1.1 and 9.2.1.2 with no modification.
  - For a 36" Type S barrier on a deck with wearing course supported by steel beams spaced greater than 12 feet apart, provide top transverse hooked #5 bars at 6" spacing over a distance of 8 feet from the joint. Lap these bars to the top transverse #6 bars at 6" called for by Table 9.2.1.2. Include 180 degree standard hooks on the edge-of-deck ends of the #5 bars.
  - For all other cases with a 36" Type S barrier, provide #5 bars at 5" spacing or  $A_s = 0.74 \text{ in}^2/\text{ft}$  for the top transverse bars over a distance of 8 feet from the joint. Include 180 degree standard hooks on the edge-of-deck ends of these bars. This can be accomplished either by providing hooked overhang bars that splice to the main transverse deck bars or by providing hooked transverse bars that run from edge to edge of the deck.
  - For the 42" Type S, where the deck consists of a 9" monolithic slab or a 7" structural slab plus a 2" concrete wearing course, include 180 degree standard hooks on the edge-of-deck ends of the top transverse bars over a distance of 9 feet from the joint. This can be accomplished either by providing hooked overhang bars that splice to the main transverse deck bars or by providing hooked transverse bars that run from edge to edge of the deck.

- For the 54" Type S, no modification is needed. Provide reinforcement per the BDM deck reinforcement Tables 9.2.1.1 and 9.2.1.2.
- Provide an edge-of-deck thickness that is equal to the deck thickness specified in BDM Tables 9.2.1.1 and 9.2.1.2 for all cases (typically 9", except for wide beam spacings). When edge-of-deck thicknesses are greater than 9", adjust the wingwall coping height to match the edge-of-deck thickness. Where the wingwall ties into a retaining wall, coordinate with the roadway designer as needed to set the retaining wall coping height to also match the edge-of-deck thickness.
- For deck overhangs that require a special design, use the following guidance for checking the extreme event limit state.
- Design collision loads  $F_{cdes}$  and  $M_{cdes}$  will be needed to complete the deck overhang design. For each barrier height  $H$ , values for  $L_c$ ,  $M_c$ , and  $R_w$  have been determined using the yield line method found in AASHTO LRFD Spec. Article A13.3.1. The TL-4 value of 54 kips for  $F_t$  was adjusted for the difference between the barrier height and height of  $F_t$  application. Then  $4/3 \cdot F_t$  was compared to  $R_w$ , and the smaller value distributed over  $L_c + H$  for end regions and  $L_c + 2H$  for interior regions. Also,  $M_c$  was adjusted when  $4/3 \cdot F_t$  governed. The results are the moments  $M_{cadj}$  and tension forces  $F_{cadj}$  given in Table 1 below.

	36" Type S		42" Type S		54" Type S	
	Exterior	Interior	Exterior	Interior	Exterior	Interior
$M_{cadj}$ (k-ft/ft)	20.5	9.4	18.8	7.8	17.4	6.7
$F_{cadj}$ (k/ft)	7.9	3.7	6.1	2.7	4.1	1.8

Table 1

In order to use these values, translate the moment  $M_{cadj}$  at the top of the deck to a moment  $M_{cdes}$  located at the center of the deck using the following method (refer to Figure 2):

$$e = M_{cadj} / F_{cadj}$$

$$F_{cdes} = F_{cadj}$$

For monolithic decks:  $M_{cdes} = F_{cdes} \cdot (e + 0.5 \cdot t_{deck})$

For decks consisting of structural slab plus wearing course:

$$M_{cdes} = F_{cdes} \cdot (e + 0.5 \cdot t_{str\ slab})$$

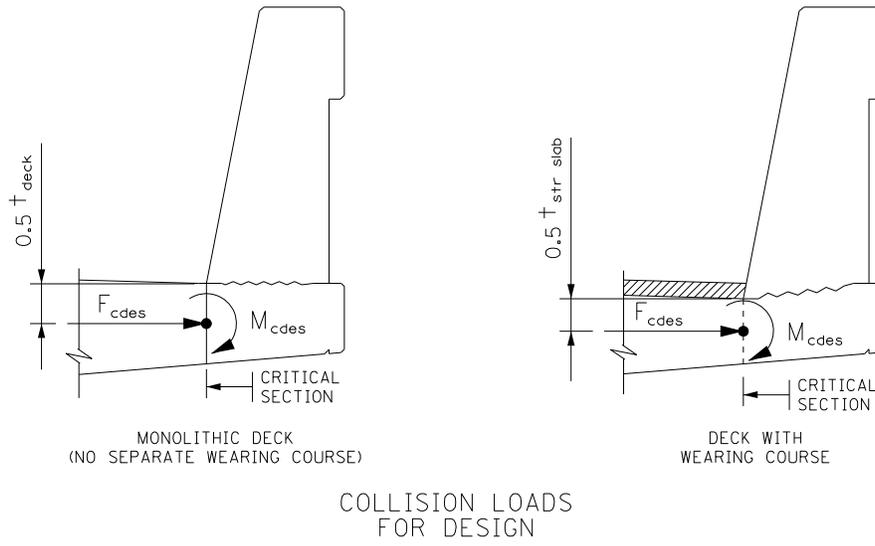
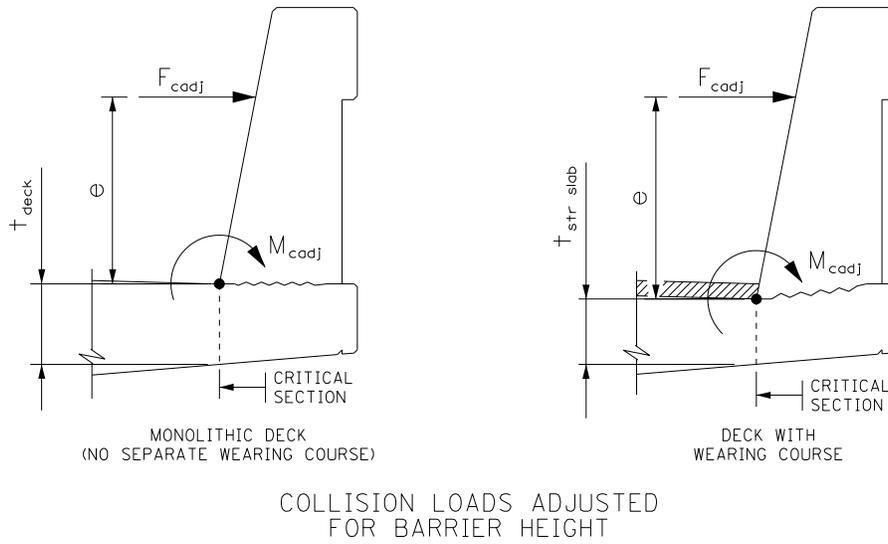


Figure 2

Use the tension force  $F_{cdes}$  and moment  $M_{cdes}$  as the collision loads for the deck overhang design.

Revision of the edge-of-deck thickness and coping height per this memo is to begin immediately for all new preliminary bridge plans.

If you have any questions, please contact Dave Dahlberg ([dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or (651) 366-4491) or me.

- cc: Kevin Western  
 Dave Dahlberg  
 Dave Conkel/Local Bridge Design Consultants  
 Colleen Lichtsinn/Bridge Design Consultants



## Memo

To: Bridge Design Engineers

From: Arielle Ehrlich   
State Bridge Design Engineer

Date: October 19, 2017

### MEMO TO DESIGNERS (2017-02): Post-Installed Anchorages for Reinforcing Bars

For bridge projects that require attachment of a new concrete element to an existing concrete element (e.g., attaching a new bridge barrier to an existing deck), adhesive anchorages with epoxy coated bars have often been used to make the connection. Recently, it was noted that adhesive manufacturers do not warrant their products for bars that have an epoxy coating, which means the design strengths reported in the literature do not apply to epoxy coated bars. Until research and testing of epoxy coated bars with adhesive anchorages is done to provide answers on this issue, use the attached table to determine the bar type, testing level, and pay item when post-installed anchorages for reinforcing bars are required for a bridge project.

Generally, the table guidance is based on the following:

- For cases where the rebar being anchored must resist significant tension and has high exposure to deicing salts and the existing concrete element has epoxy coated rebar, stainless steel rebar is required with a high level of testing that confirms adequate tensile resistance.
- For cases where the rebar being anchored must resist significant tension and has low exposure to deicing salts or the existing concrete element has uncoated rebar, uncoated rebar is required with a high level of testing that confirms adequate tensile resistance.
- For cases where the rebar being anchored must resist little or no tension, an epoxy coated rebar is required with a low level of testing that confirms adequate placement procedures.

For all other bars in the new concrete element (those that are not being anchored), provide epoxy coated rebar regardless of whether the existing element contains uncoated or epoxy coated bars.

Use of this guidance is to begin immediately for all repair projects in the final design phase.

If you have any questions, please contact Dave Dahlberg ([dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us)) or (651) 366-4491) or me.

cc: Kevin Western  
Dave Dahlberg  
Dave Conkel/Local Bridge Design Consultants  
Colleen Lichtsinn/Bridge Design Consultants

**Post-Installed Anchorages for Reinforcing Bars - Bar Type and Testing Level**

Type of Construction	Existing Structure Rebar Type	Anchored Rebar Location	Anchored Rebar Type	Anchorage ② Testing Level	Pay Item
Barrier anchored to bridge deck or approach panel	Epoxy coated bars in deck or approach panel	Front face of barrier	Stainless	H	2433.502 ANCH TYPE REINF BARS (STAINLESS TYPE H) EACH
		Back face of barrier ①	Epoxy coated	L	2433.502 ANCH TYPE REINF BARS (TYPE L) EACH
	Uncoated bars in deck or approach panel	Front face of barrier	Uncoated	H	2433.502 ANCH TYPE REINF BARS (TYPE H) EACH
		Back face of barrier ①	Epoxy coated	L	2433.502 ANCH TYPE REINF BARS (TYPE L) EACH
Bridge barrier end posts anchored to abutments	All types	Any location	Uncoated	H	2433.502 ANCH TYPE REINF BARS (TYPE H) EACH
Pier crash struts anchored to pier footings	Epoxy coated bars in columns	Vertical dowels in tension	Stainless	H	2433.502 ANCH TYPE REINF BARS (STAINLESS TYPE H) EACH
	Uncoated bars in columns	Vertical dowels in tension	Uncoated	H	2433.502 ANCH TYPE REINF BARS (TYPE H) EACH
Pier cap repairs	All types	Bars resisting significant tension	Uncoated	H	2433.502 ANCH TYPE REINF BARS (TYPE H) EACH
		Other bars	Epoxy coated	L	2433.502 ANCH TYPE REINF BARS (TYPE L) EACH
Infill walls for strengthening piers	All types	Any location	Epoxy coated	L	2433.502 ANCH TYPE REINF BARS (TYPE L) EACH
Abutment paving bracket reconstruction	All types	Any location	Epoxy coated	L	2433.502 ANCH TYPE REINF BARS (TYPE L) EACH
Abutment end block reconstruction	All types	Any location	Epoxy coated	L	2433.502 ANCH TYPE REINF BARS (TYPE L) EACH
Prestressed concrete beam repairs	All types	Any location	Epoxy coated	L	2433.502 ANCH TYPE REINF BARS (TYPE L) EACH
All other situations	To be determined on a case-by-case basis				

① Install back face bars at twice the front bar spacing, not to exceed 2'-0"

② H = High level of testing to confirm adequate resistance to tensile loads

L = Low level of testing to confirm adequate placement procedures

Refer to special provision SB2018-2433.8 for testing requirements

## Memo

**Date:** 12/20/2018

**To:** Bridge Design Engineers

**From:** Arielle Ehrlich   
State Bridge Design Engineer

### **RE: Memo to Designers #2018-01: New 30MH, 35MH, and 40MH Prestressed Concrete Beams**

Recently, MnDOT began investigating prestressed beam shapes that could be used more effectively in span ranges of 75 to 115 ft. The MH series shape that has been developed incorporates elements of several prestressed concrete beams currently in use around the country. Three beam depths have been chosen, which are designated as 30MH, 35MH, and 40MH. Based on discussions with fabricators related to obtaining forms, we have agreed to start specifying the 30MH and 35MH beams for lettings July 1, 2019 and later. The 40MH beams may be used for lettings November 1, 2019 and later.

Attached are Figures 5.4.6.1 and 5.4.6.2 that will be updated in the LRFD Bridge Design Manual (BDM) showing beam section properties and the preliminary beam selection chart for the RB, M, MH, and MN series. The 27M and the 36M beams will still be available and should continue to be used where appropriate.

Below is a listing of the Standard Plans and B-Details that were developed or modified for the new shape:

- 5-397.501 30MH Prestressed Concrete Beam
- 5-397.502 35MH Prestressed Concrete Beam
- 5-397.503 40MH Prestressed Concrete Beam
- B303 Sole Plate
- B307 Bearing Pad Restraint
- B309 Tapered Bearing Plate Assembly
- B310 Curved Plate Bearing Assembly - Fixed
- B311 Curved Plate Bearing Assembly - Expansion
- B403 Steel Intermediate Bolted Diaphragm
- B814 Concrete End Diaphragm – Parapet Abutment

These standards are currently approved and available for use.

Several criteria currently listed in the BDM for prestressed girders have been changed due to the development of the MH series and are discussed below.

## Intermediate Diaphragms

Intermediate diaphragms are not required for 30MH and 35MH beams. The 40MH will follow BDM Article 5.4.1 guidelines for intermediate diaphragm spacing.

## Beam End Dimensions

For MH prestressed beams, follow the guidance given in BDM Article 5.4.1 for RB, M, and MN shapes.

## Camber Prediction

For MH prestressed beams, follow the guidance given in BDM Article 5.4.5 for RB, M, and MN shapes and the use of camber multipliers.

## Overhang Criteria

Overhang criteria remains the same as is shown in Figure 9.2.1 of the BDM.

## Bearings

The majority of guidance given in BDM Section 14 regarding bearings applies when using MH series prestressed beams. However, the minimum elastomeric pad size for MH beams is 12 inches (length A) by 30 inches (width B). In addition, BDM Tables 14.7.1, 14.7.2, and 14.7.3 have been revised to include standard B310 and B311 bearing dimensions for the MH and MW series beams. The revised tables are included as attachments to this memo.

## Material Properties

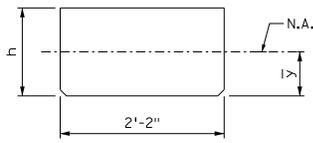
Concrete, prestressing strand, and mild reinforcement properties remain as specified in the BDM. The attached charts for span length and beam spacing assume a concrete release strength ( $f'_{ci}$ ) of 7.5 ksi and a final concrete strength ( $f'_c$ ) of 9 ksi. With approval of the State Bridge Design Engineer, final concrete strengths of 10 ksi may be permitted for the MH series beams.

For questions about this policy, please contact Dave Dahlberg ([dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us)) or (651) 366-4491) or Arielle Ehrlich ([arielle.ehrlich@state.mn.us](mailto:arielle.ehrlich@state.mn.us)) or (651) 366-4506).

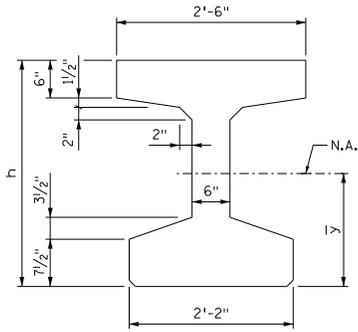
cc: K. Western  
D. Dahlberg  
P. Rowekamp  
C. Lichtsinn/Design Consultants  
D. Conkel/Local Consultants

Attachments: 5-29.1, 5-29.2, 14-15, 14-16 of the LRFD Bridge Design Manual

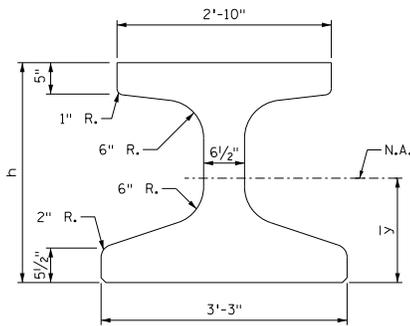
*An Equal Opportunity Employer*



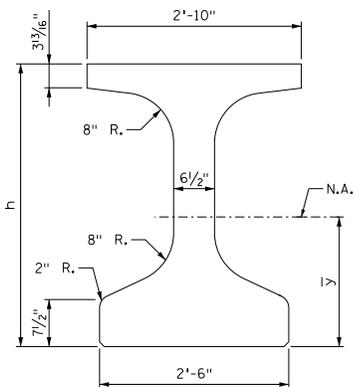
RECTANGULAR BEAM



"M" SERIES I-BEAM



"MH" SERIES I-BEAM



"MN" SERIES I-BEAM

DESIGN ASSUMPTIONS FOR PRESTRESSED CONCRETE BEAM CHART:

2017 AASHTO LRFD Bridge Design Specifications, 8th Edition.

HL-93 Live Load

Beam Concrete:  $f'_c = 9.0$  ksi  $f'_{ci} = 7.5$  ksi  $w_{bm} = 0.155$  kips/ft<sup>3</sup>

$$E_c = 1265\sqrt{f'_c} + 1000 \text{ ksi}$$

Deck Concrete:  $f'_c = 4.0$  ksi  $E_c = 3987$  ksi

$w_c = 0.145$  kcf for  $E_c$  computation

$w_c = 0.150$  kcf for dead load computation

0.6" diameter low relaxation strands,  $E_s = 28,500$  ksi

$f_{pu} = 270$  ksi with initial pull of  $0.75 f_{pu}$

Simple supports with six beams and deck without wearing course.  
Deck carries two Type S Barriers with no sidewalk or median.  
Skew = 0 degrees.

Effective deck thickness is total deck thickness minus 1/2" of wear.

1 1/2" stool height used for composite beam section properties.  
2 1/2" average stool height used for dead load calculations.

Rail dead load applied equally to all beams.  
Dead load includes 0.020 ksf future wearing course.

Approximate long term losses are used per LRFD 5.9.5.3.

Service Concrete Tensile Stress Limits:

After Initial Losses:  $0.094\sqrt{f'_{ci}} \leq 0.2$  ksi

After All Losses:  $0.19\sqrt{f'_c}$

Beam Properties

BEAM	h (in)	AREA (in <sup>2</sup> )	W ① (lb/ft)	$\bar{y}$ (in)	I (in <sup>4</sup> )	S <sub>B</sub> (in <sup>3</sup> )	A <sub>c</sub> ② (in <sup>2</sup> )
14RB	14	364	392	7.00	5,945	849	312
18RB	18	468	504	9.00	12,640	1,404	364
22RB	22	572	616	11.00	23,070	2,097	416
27M	27	516	555	13.59	43,080	3,170	296
30MH	30	639	688	13.66	70,416	5,155	403
35MH	35	672	723	15.85	105,570	6,661	419
36M	36	570	614	17.96	93,530	5,208	323
40MH	40	704	758	18.07	149,002	8,246	435
MN45	45	690	743	20.58	178,780	8,687	427
MN54	54	749	806	24.63	285,230	11,580	457
MN63	63	807	869	28.74	421,750	14,670	486

① Based on 155 pounds per cubic foot.

② Based on a 9" slab with 1/2" of wear and 1 1/2" stool. See LRFD 5.8.3.4.2 for A<sub>c</sub> definition.

Figure 5.4.6.1  
Precast Prestressed Concrete Beam Data (RB, M, MH, MN)

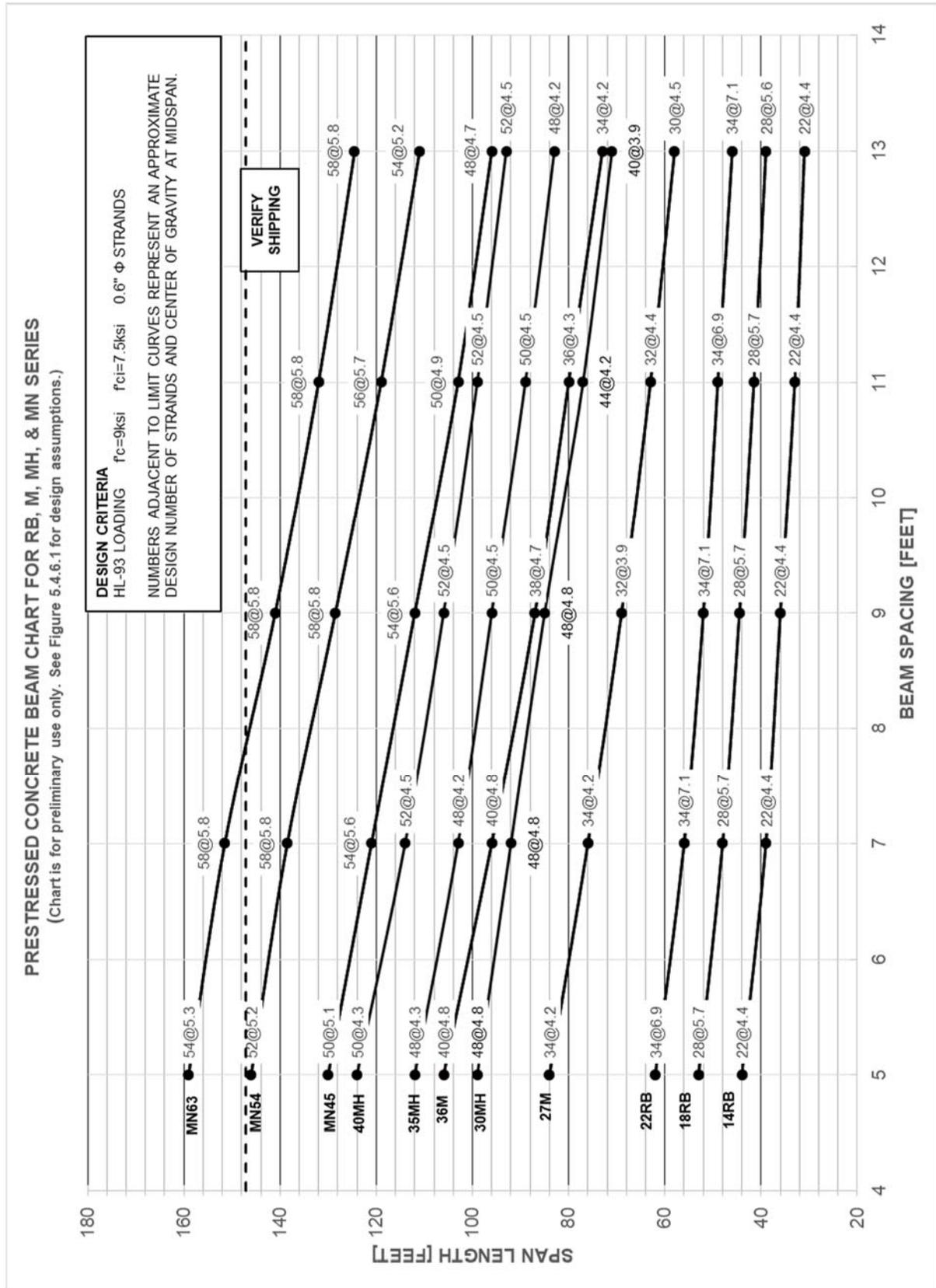


Figure 5.4.6.2

**Table 14.7.1**  
**Fixed Curved Plate Bearing Assembly for Prestressed Concrete Beams (B310)**

Beam Series	Max Service DL+LL (kips)	Bearing Pad Size (in)		Plain Pad Thickness (in)	Shape Factor	Bearing Plate Size (in) ②			Curved Plate Size (in) ②			Min Radius (in)
		A	B			C	E	F	G	H	J	
RB, M, and MN	253	12	24	1/2	8.0	14	①	1 1/2	4 1/2	26	1 1/4	16
	295	14	↓	↓	8.8	16	↓	↓	6	↓	↓	↓
	337	16	↓	↓	9.6	18	↓	2	↓	↓	↓	↓
	380	18	↓	3/4	6.9	20	↓	↓	8	↓	↓	↓
	422	20	↓	↓	7.3	22	↓	2 1/4	↓	↓	↓	20
MH	316	12	30	1/2	8.6	14	47	1 1/2	4 1/2	32	1 1/4	16
	369	14	↓	↓	9.6	16	↓	↓	6	↓	↓	↓
MW	270	16	36	1/2	11.1	18	47	1 1/2	4 1/2	38	1 1/4	16
	350	↓	↓	↓	↓	↓	↓	↓	6	↓	↓	↓
	506	↓	↓	↓	↓	↓	↓	↓	8	↓	↓	↓
	570	18	↓	↓	12.0	20	↓	2	8	↓	↓	↓

① 34" for all "RB" and "M" series beams.

38" for all "MN" series beams.

② Plates are conservatively designed for 1.75 · (Max Service DL+ LL).

**Table 14.7.2**  
**Expansion Curved Plate Bearing Assembly for Prestressed Concrete Beams (B311)**

Beam Series	Max Service DL+LL (Kips)	Bearing Pad Size (in)		Laminate Thickness (in)	Max Number of Laminates ①	Shape Factor	Bearing Plate Size (in) ②			Curved Plate Size (in) ②			Min Radius (in)
		A	B				C	E	F	G	H	J	
RB, M, and MN	300	12	24	1/2	7	8.0	14	27	1 1/2	4 1/2	26	1 1/4	16
	360	12	↓	↓	7	8.0	14	↓	1 3/4	↓	↓	↓	↓
	420	14	↓	↓	8	8.8	16	↓	↓	6	↓	↓	19
MH	395	12	30	1/2	7	8.6	14	33	1 1/2	4 1/2	32	1 1/4	16
MW	270	16	36	3/4	6	7.4	18	39	1 1/2	4 1/2	38	1 1/4	16
	350	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
	480	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
	630	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓

① See Table 14.7.3 for determination of required number of laminates.

② Plates are conservatively designed for 1.75 · (Max Service DL+ LL).

**Table 14.7.3**  
**Elastomeric Bearing Pad Thickness for Expansion Curved Plate**  
**Bearing Assembly for Prestressed Concrete Beams (B311) ①②**

Interior Laminate Thickness (in)	D (in) ③	Number of Laminates	Total Elastomer Thickness, $h_{rt}$ (in) ③	Maximum Movement $\Delta_s$ (in) ④
1/2"	1 1/4	1	1	1/2
	1 7/8	2	1 1/2	3/4
	2 1/2	3 ⑤	2	1
	3 1/8	4	2 1/2	1 1/4
	3 3/4	5	3	1 1/2
	4 3/8	6 ⑤	3 1/2	1 3/4
	5	7	4	2
	5 5/8	8	4 1/2	2 1/4
3/4"	1 1/2	1	1 1/4	7/8
	2 3/8	2	2	1
	3 1/4	3	2 3/4	1 3/8
	4 1/8	4	3 1/2	1 3/4
	5	5	4 1/4	2 1/8
	5 7/8	6	5	2 1/2

① Table is based on requirements of *AASHTO LRFD Bridge Design Specs*. Art. 14.7.6.3.4:

$$h_{rt} > 2\Delta_s$$

Engineer must also check that the minimum compressive load requirement (discussed in Article 14.3.3.3.1) is satisfied. Specifically:

$$P_{min} \geq 5 \cdot G \cdot A_{pad} \cdot \frac{\Delta_u}{h_{rt}}$$

where  $P_{min}$  is the minimum factored load ( $0.9 \cdot DC + 1.75 \cdot LL_{min}$ ),  $G$  is equal to the maximum shear modulus value (0.200 ksi),  $A_{pad}$  is the plan area of the bearing pad, and  $\Delta_u$  is the movement of the bearing pad from the undeformed state using a 75°F temperature.

② Engineer must also check the elastomeric bearing pad for compression deflection based on the requirements from *AASHTO LRFD Bridge Design Specifications* Articles 14.7.6.3.3 and 14.7.5.3.6.

③ Pad thickness  $D$  includes  $h_{rt}$  and 1/8" steel reinforcement plates. Total elastomer thickness  $h_{rt}$  includes interior laminates plus 1/4" cover layers.

④ Maximum movement  $\Delta_s$  is the movement of the bearing pad from the undeformed state to the point of maximum deformation. Use a 75°F temperature change with a 1.3 load factor for calculation of maximum movement.

⑤ For "RB", "M", and "MN" series prestressed beam expansion elastomeric bearings, the number of laminates has been standardized for the movements that are most often encountered.

- If  $\Delta_s \leq 1.00"$ , use 3 - 1/2" laminates.
- If  $1.00" < \Delta_s \leq 1.75"$ , use 6 - 1/2" laminates.

## Memo

Date: 1/30/2019

To: Bridge Design Engineers

From: Arielle Ehrlich   
State Bridge Design Engineer

### RE: Memo to Designers #2019-01: Temporary Portable Precast Concrete Barrier on Bridges

#### Introduction

This guidance supersedes the guidance given in Memo to Designers #2011-03. With the release of this memo, Memo to Designers #2011-03 is officially rescinded.

This memo applies to MnDOT Standard Plate 8337, Temporary Portable Precast Concrete Barrier Type "F". Barrier segments are connected together end-to-end with connection pins and are either anchored or unanchored.

Temporary portable precast concrete barrier can be used to protect vehicles from severe hazards (e.g. - large vertical drop-offs, fixed objects), provide a positive separation between traffic and work zones, or to separate opposing lanes of traffic. The *Temporary Barrier Guidance Manual* (TBGM) has been created to provide requirements for use of temporary barrier on MnDOT projects. The manual can be downloaded at:

<http://www.dot.state.mn.us/trafficeng/workzone/wzmanual.html>

It includes guidance on work zone clear zones, temporary barrier length of need, deflection distance, anchorage types, etc. This Memo to Designers #2019-01 provides guidance specific to temporary barrier located on bridges.

#### Deflection Distance Requirements

Good barrier performance is dependent on the provision of adequate deflection distance. Deflection distance is defined as the distance the barrier travels laterally during impact. It is dependent on the speed at impact and whether the barrier is anchored or unanchored. For barrier on bridges, the distance from the edge of deck to the edge of the temporary barrier on its non-traffic side must be at least equal to the deflection distance. Refer to the tables in Article 3.4 and notes in Article 3.4.1 of the TBGM for all of the deflection distance requirements for bridges. Note that the deflection distances given in Tables 3-2 and 3-3 are minimum values required for the safety of vehicle occupants. If additional deflection distance is required by the roadway designer, carry that distance across the bridge.

## Anchor Requirements

The decision to use anchored or unanchored barrier is dependent on the bridge width and staging requirements. The following anchor requirements must be met when anchoring temporary barrier to a bridge deck (refer to Bridge Details Manual Part I, B920 for additional information and requirements):

- For each barrier segment, install three, 1½” diameter anchor rods (MnDOT Spec. 3385 Type A) on the traffic side only.
- For bridge decks with top mat reinforcement and sound concrete, specify adhesive anchors with 5½” minimum embedment. Specify a maximum hole depth equal to 1½ inches less than the slab depth to help ensure that the bottom of the slab does not spall or fracture during hole drilling.
- For adhesive anchors, specify a minimum characteristic bond strength in uncracked concrete of 1.0 ksi and a proof load of 7.0 kips. See the standard special provision SB2018-2433.8 D for additional testing requirements.
- Utilize through-deck anchoring when any of the following is true:
  - Deck concrete is in poor condition. (Deck concrete is in Condition State 3 or 4 in areas where anchors are to be located.)
  - Deck does not have a top mat of reinforcement.
  - Deck has a bituminous overlay.

The Regional Bridge Construction Engineer will confirm adequacy for installations on existing bridges.

Refer to the notes in Article 3.4.1 of the TBGM for additional anchoring requirements.

## Bridge Plan and Special Provision Requirements

For bridges with unanchored temporary barrier, the pay item for temporary portable precast concrete barrier is included in the roadway plan. Coordinate with the roadway designer to confirm that the assumed layout of the temporary barrier on the bridge agrees with the final layout shown in traffic control plans.

For bridges with anchored temporary barrier, the pay item for temporary portable precast concrete barrier is included in the roadway plan, and all work associated with anchorages is incidental to that pay item. In the bridge plan, include standard detail B920 (Bridge Details Manual Part I), which shows anchorage details for the temporary barriers. Also include standard special provision SB2018-2433.8 D in the special provisions for the bridge, which covers: anchorage installation, anchorage testing, anchorage removal, and repair of the bridge deck. Coordinate with roadway designer to confirm that the assumed layout of the temporary barrier on the bridge agrees with the final layout shown in traffic control plans.

Deployment length and anchorage requirements past the ends of the bridge are determined by the roadway designer and shown in the traffic control plans. When temporary or permanent sheet piling (or other type of wall) is included in the bridge plan, alert the roadway designer that there is a vertical drop-off hazard located off the bridge that must be considered when determining the temporary barrier requirements, including the deflection distance (refer to Article 3.4.1 of the TBGM).

For questions about this policy, please contact Dave Dahlberg ([dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651-366-4491) or Arielle Ehrlich ([arielle.ehrlich@state.mn.us](mailto:arielle.ehrlich@state.mn.us) or 651-366-4506).

cc: K. Western  
D. Dahlberg  
P. Rowekamp  
C. Lichtsinn/Design Consultants  
D. Conkel/Local Consultants

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## Transmittal Notice 2019-01

**Date:** 1/30/2019

**Distribution:** MnDOT Bridge Office Web Site

**Issued by:** MnDOT Bridge Office

### Subject: MnDOT LRFD Bridge Design Manual Update

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF at:

<http://www.dot.state.mn.us/bridge/>

Check this Web site regularly for updates or to sign up for email update alerts.

#### Instructions (for two-sided printing):

1. Remove from the manual:
  - Title Page
  - Table of Contents pages iii, iv, xi, and xii
  - Entire Section 3
  - Entire Section 15
  - Memo to Designers (2011-03) Interim Guidance for Installation of Temporary Barriers on Bridges and Approach Panels
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages iii, iv, xi, and xii
  - Entire Section 3
  - Memo to Designers #2019-01: Temporary Portable Precast Concrete Barriers on Bridges

#### Update Summary

Revisions in the "JANUARY 2019" update include:

- Section 3 – Changed the Word file format from .doc to .docx, which caused minor shifting of some text locations in the .pdf version. As a result, the entire Section is being republished in its entirety. Changes from the previous version are denoted by a single vertical revision bar located in the right margin.
- Article 3.4.1 – Changed double truck requirements from text to equation form for clarification. Also added guidance for evaluation of existing bridges.
- Article 3.5 – Added further explanation of "check flood for scour" and a figure for clarification.
- Article 3.10.1 – Revised definition of a typical bridge.

- Section 15 on bridge load rating is being deleted. For guidance on load rating, refer to the *MnDOT Bridge Load Rating and Evaluation Manual*, found at: <http://www.dot.state.mn.us/bridge/datamanagement.html>
- Memo to Designers (2011-03) Interim Guidance for Installation of Temporary Barriers on Bridges and Approach Panels is not applicable anymore, so has been rescinded. Memo to Designers #2019-01: Temporary Portable Precast Concrete Barriers on Bridges, replaces the old memo.

For **technical questions** regarding this transmittal contact Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651-366-4491.



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Date: December 20, 2018

## Transmittal Notice 2018-03

DISTRIBUTION	MnDOT Bridge Office Web Site
MANUAL	LRFD Bridge Design Manual
SUBJECT	Memo to Designers #2018-01: New 30MH, 35MH, and 40MH Prestressed Concrete Beams

An update to the MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

### INSTRUCTIONS:

(for two-sided printing)

1. Remove from the manual:
  - Title Page
  - Table of Contents pages xi and xii
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages xi and xii
  - Memo to Designers (2018-01)

Note: The "DECEMBER 2018" update contains the following:

- Addition of Memo to Designers (2018-01), which contains guidance regarding new MH Series prestressed concrete beams.

Direct any *technical questions* regarding this transmittal to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.

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October 30, 2018

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
<b>TRANSMITTAL NOTICE (2018-02)</b>		<b>ISSUED BY</b>	Bridge Office
<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents Section 2 General Design and Location Features		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

(for two-sided printing)

1. Remove from the manual:
  - Title Page
  - Table of Contents pages i through iv
  - Entire Section 2
  
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages i through iv
  - Section 2

Note: Because of the numerous changes spread throughout the section, Section 2 is being republished in its entirety. Changes from the previous version are denoted by a single vertical revision bar located in the right margin. The "OCTOBER 2018" update includes the following revisions:

- Article 2.1.1 - **General Criteria** was revised for consistency with performance based practical design (PBPD) guidance.
- Article 2.1.2 - **Bridge Width Criteria** was updated for consistency with PBPD guidance.
- Article 2.1.2 - **Cross Slopes on Bridges** was revised to match current practice.

- Article 2.1.2 – In **Waterway Sections Under Bridges**, added a note on wildlife passage benches.
- Deleted Article 2.1.4 for consistency with PBPD guidance, except retained and renumbered railroad clearances figure.
- Article 2.3.1 – In **Preparation of Preliminary Plans**, added paragraph on accelerated bridge construction (ABC).
- Article 2.3.2 – Revised **Abutment Types** guidance to include information regarding abutment height and relative cost.
- Article 2.3.2 – Under **Pier Types**, added general guidance.
- Article 2.4.1.2 – Revised concrete mix type for pedestrian bridge decks.
- Article 2.4.1.6.1 – Revised article title.
- Article 2.4.1.6.2 – Revised Figure 2.4.1.6.2.1 and added guidance for **Utilities Near MSE Walls**.
- Figures 2.5.1.3.1, 2.5.1.3.2, 2.5.1.3.3, 2.5.1.3.9, and 2.5.3.3 – Revised joint sealer spec number.
- Figure 2.5.1.3.9 – Added missing V-strip.
- Article 2.5.2.1 – Corrected pay item number for "Reconstruct Paving Bracket and Wall".
- Deleted Article 2.5.3 on pavement joints as they are not used and/or are out of date.
- Appendix 2-C – Revised some of the standard plan notes to reflect current practice.
- Appendix 2-D – Added new quantity note.
- Corrected web links.
- Made other minor changes for clarification.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.



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September 25, 2018

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents Section 9 Decks and Deck Systems		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents pages vii through viii
  - Entire Section 9
  
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages vii through viii
  - Section 9

**Note:** Due to the numerous changes, Section 9 is being republished in its entirety without any revision bars in the right margin. The "SEPTEMBER 2018" update includes the following revisions to Section 9:

- Removed old bridge deck protection policy and replaced with a reference to the new policy in BDM Section 2.
- Added language that deck drains and drainage systems are strongly discouraged on bridges. Also revised deck drain guidance for clarification.
- Added wording to deck design guidance that epoxy coated rebar is default type, and added paragraphs on stainless and GFRP bars.

- Put deck design requirements in a bulleted list for ease of use by designers.
- Added clarification on basis of distribution reinforcement for bridges with varying beam spacing.
- Added that decks not meeting deck reinforcement table assumptions require a special design.
- Revised guidance on deck overhang design to be consistent with Memo to Designers (2017-01).
- Changed standard curb height from 8" to 6".
- Under deck placement sequence guidance: added requirement to analytically determine construction joint location for long bridges, added to consider girder rotation, and added to consider orienting deck pours in uphill direction.
- Made additions to deck reinforcement table assumptions to get a more complete list.
- Revised deck reinforcement tables to match with AASHTO code changes.
- Updated deck design examples.
- Revised figures throughout the section based on current office practices including: Change from F-barrier to Type S barrier, change in edge of deck detailing for decks with a wearing course, change from metric to US customary rebar designation, changes to parapet and curb details, clarifications to notes and call-outs, changes to design example figures for clarification, addition of figures to improve design examples.
- Made numerous changes from passive voice to active voice.
- Made other minor changes for clarification.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.



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December 11, 2017

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents Section 2: General Design and Location Features		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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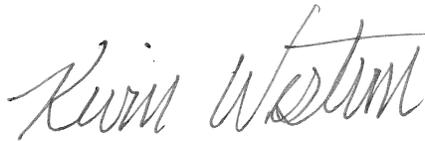
1. Remove from the manual:
  - Title Page
  - Table of Contents pages i through iv
  - Entire Section 2
  - Memo to Designers (2014-02)
  
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages i through iv
  - Section 2

Note: The previous revision to Section 2 contained 3 margin revision bars. For this revision, all vertical margin revision bars have been removed and the section is being republished in its entirety. The "DECEMBER 2017" update includes the following revisions:

- For bridge width criteria, references Technical Memo 12-14-B-03. After memo expires, directs users to contact the Preliminary Bridge Plans Engineer.
- Revisions to bridge pedestrian walkway and shared-use path width criteria.
- Incorporated bridge vertical clearance requirements for underpasses from the expired Technical Memo 11-16-B-07.
- Updated time schedule for preliminary bridge plan.

- Updated table regarding preliminary bridge plan distribution.
- Updated guidance regarding FHWA oversight.
- Revised buried utility guidance to include sheet pile walls and soldier pile walls as well as spread footings.
- Revised buried utility guidance to include gas lines as wet utilities.
- Revised bridge deck protection policy.
- Updated general plan & elevation and typical cross-section figures.
- Additional guidance is given regarding pay items.
- Changes to guidance on use of standard bridge details and bridge standard plans within bridge plan sets.
- Changes to guidance on revised plan sheets.
- Changes to rebar and concrete mix callouts in Article 2.5 reconstruction figures.
- Removed figures on railing reconstruction and expansion joint Type X reconstruction.
- Addition of details and pay items for abutment paving bracket repair.
- Revised and updated guidance on bridge type numbers.
- Revisions to the Standard Plan Notes.
- Revisions to the Standard Summary of Quantities Notes.
- Removal of lists of Bridge Special Provisions, Bridge Details I & II, Bridge Standard Plans, Roadway Standards for Retaining Walls, Bridge Pay Items.
- Removal of MEMO TO DESIGNERS (2014-02), as guidance given regarding informational quantities has been added to BDM Art. 2.4.2.2.
- Numerous minor changes.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.



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November 27, 2017

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
<b>TRANSMITTAL NOTICE (2017-03)</b>		<b>ISSUED BY</b>	Bridge Office
<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents Section 14: Joints and Bearings		

An update to the MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents pages ix and xii
  - Entire Section 14
  
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages ix and xii
  - Section 14

**Note:** The "NOVEMBER 2017" update includes the following revisions:

- Updated the elastomeric bearing guidance and tables to reflect material hardness change from 55 to 60 durometers.
- Updated the elastomeric bearing guidance and tables to standardize the number of laminates for prestressed concrete beam bearings.
- Updated the elastomeric bearing guidance to match AASHTO changes regarding the minimum compressive load.
- Replaced the reference to knock-off studs with new reference to 3/8" x 3/8" bar.
- Clarified guidance on design of the bearing plate and curved plate.
- Added notes explaining our current practice of using cotton duck pads rather than plain elastomeric pads.

- Replaced article on pot bearings with similar article on disc bearings.
- Revised guidance, tables, and examples regarding curved plate and bearing plate design from the previous method of designing for max service stress equal to  $0.55F_y$  to LRFD strength design for flexure limited to  $F_y$ .
- Added a bullet to the bridge fixity guidance regarding bridges on a slope tending to move downhill.
- Added note to consider creep movement for post-tensioned concrete bridges.
- Added guidance on where a 1/8" plain pad is to be used.
- Added note stating that pot bearings are not to be used on roadway bridges.
- Added guidance for prestressed concrete beam bridges on slopes  $> 3\%$ .
- Updated the fixed elastomeric bearing example.
- Updated the expansion elastomeric bearing example.
- Made numerous editorial changes.
- Changed some of the existing guidance from passive to active voice.

Direct all **technical questions** regarding this transmittal to:

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[dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us)  
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October 19, 2017

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents MEMO TO DESIGNERS (2017-02): Post-Installed Anchorages for Reinforcing Bars		

An update to the MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

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1. Remove from the manual:
  - Title Page
  - Table of Contents pages xi and xii
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages xi and xii
  - Memo to Designers (2017-02)

Note: The "OCTOBER 2017" update includes the following revisions:

- Addition of MEMO TO DESIGNERS (2017-02), which contains guidance regarding bar type, testing level, and pay item when post-installed anchorages for reinforcing bars are required for a bridge project.

Direct all **technical questions** regarding this transmittal to:

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March 28, 2017

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
<b>TRANSMITTAL NOTICE (2017-01)</b>		<b>ISSUED BY</b>	Bridge Office
<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents MEMO TO DESIGNERS (2017-01): Edge-of-Deck Thickness on Bridges and Wall Coping Height		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in Adobe PDF (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This website should be checked regularly for updates.

#### **INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents pages xi and xii
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages xi and xii
  - MEMO TO DESIGNERS (2017-01)

Note: The "MARCH 2017" update includes the following revisions to the BDM:

- Addition of MEMO TO DESIGNERS (2017-01), which contains changes to guidance regarding the edge-of-deck thickness and wall coping height.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.



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December 9, 2016

Table with 4 rows and 3 columns. Row 1: DISTRIBUTION: MnDOT Bridge Office Web Site. Row 2: TRANSMITTAL NOTICE (2016-04), ISSUED BY Bridge Office. Row 3: MANUAL LRFD Bridge Design Manual, DEVELOPED BY Bridge Office. Row 4: SUBJECT Title Page, Table of Contents, MEMO TO DESIGNERS (2016-01): Single Slope Barrier (Type S) Bridge Standards.

The MnDOT Bridge Office LRFD Bridge Design Manual (BDM) is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This Web site should be checked regularly for updates.

INSTRUCTIONS:

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- 1. Remove from the manual:
- Title Page
- Table of Contents pages xi and xii
- MEMO TO DESIGNERS (2011-02)
2. Print and insert in the manual:
- Title Page
- Table of Contents pages xi and xii
- MEMO TO DESIGNERS (2016-01)

Note: The "DECEMBER 2016" update includes the following revisions to the BDM:
- Addition of MEMO TO DESIGNERS (2016-01), which contains design and implementation guidance for the new single slope barrier (Type S) bridge standards.
- Removal of MEMO TO DESIGNERS (2011-02), which has been superceded by the guidance found in MEMO TO DESIGNERS (2016-01).

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

Handwritten signature of Kevin Western

Kevin Western
State Bridge Engineer





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August 10, 2016

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents Section 1: Introduction Section 3: Loads and Load Factors Section 4: Structural Analysis and Evaluation Section 5: Concrete Structures Section 8: Wood Structures Section 11: Abutments, Piers, and Walls Memos		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents pages iii through xii
  - Pages 1-9 and 1-10
  - Entire Section 3
  - Pages 4-5 and 4-6
  - Pages 5-1, 5-2, and 5-9 through 5-14
  - Pages 8-117 and 8-118
  - Pages 11-7, 11-8, 11-13, and 11-14
  - Remove Memo to Designers (2005-02), (2005-03), and (2006-01)
  
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages iii through xii
  - Pages 1-9 and 1-10
  - Entire Section 3
  - Pages 4-5 and 4-6
  - Pages 5-1, 5-2, 5-9 through 5-14, and 5-137 through 5-146
  - Pages 8-117 and 8-118
  - Pages 11-7, 11-8, 11-13, and 11-14

Note: In the "AUGUST 2016" update, some minor errors are being corrected, some clarifications are being added, and some guidance is being revised. Changes include:

- Section 1: Correction regarding when a structure is assigned a bridge number.
- Section 3: Updates regarding wind loads, correction to Table 3.3.1 regarding unit weight of compacted fill on box culverts, clarification on double truck modification, and renumbering of some articles.
- Section 4: Correction of a table reference.
- Section 5: Correction to Table 5.1.1.1 regarding concrete mix designation, update to reinforcing bar development lengths and splices, hooked bar development lengths, and minimum bar spacing. Also added appendix containing rebar lap splice guide.
- Section 8: Correction of 2 subheadings.
- Section 11: Correction of Figure 11.1.1.1a to show keyways and correction of error regarding integral abutment stem dowel embedment.
- Removal of obsolete Designer Memos.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.



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July 7, 2016

Table with 4 columns: TRANSMITTAL NOTICE (2016-02), ISSUED BY, MANUAL, DEVELOPED BY, SUBJECT. Content includes 'MnDOT Bridge Office Web Site', 'Bridge Office', 'LRFD Bridge Design Manual', and 'Section 11: Abutments, Walls, and Piers'.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This Web site should be checked regularly for updates.

INSTRUCTIONS:

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- 1. Remove from the manual: Title Page, Table of Contents pages vii through xii, Entire Section 11, Bridge Designer Memo #2007-01
2. Print and insert in the manual: Title Page, Table of Contents pages vii through xii, Section 11

Note: Due to the numerous changes, Section 11 is being republished in its entirety. The "JULY 2016" update includes the following revisions: Revised abutment, retaining wall, and pier design examples. Addition of updated pier protection policy to Section 11 and removal of outdated designer memo regarding pier protection policy. Revisions to design and detailing guidance for abutments, wingwalls, retaining walls, and piers.

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

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May 2, 2016

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents Section 8: Wood Structures		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

(for two-sided printing)

1. Remove from the manual:
  - Title Page
  - Table of Contents pages v through xii
  - Entire Section 8
2. Print and insert in the manual:
  - Title Page
  - Table of Contents pages v through xii
  - Section 8

Note: Section 8 is being republished in its entirety due to the many changes. This section has been greatly expanded from the 2003 version to include recent updates on wood materials/products, preservatives, decks, detailing, timber vehicular railings, and several new and updated design and load rating examples that meet the current AASHTO LRFD Bridge Design Specs and the AASHTO Manual for Bridge Evaluation.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.

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Minnesota Department of Transportation

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Fax No: 651/366-4497

August 10, 2015

Table with 4 columns: DISTRIBUTION, TRANSMITTAL NOTICE (2015-03), MANUAL, SUBJECT. Includes details on MnDOT Bridge Office Web Site, LRFD Bridge Design Manual, and subject matter like Title Page and Section 5: Concrete Structures.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This Web site should be checked regularly for updates.

INSTRUCTIONS:

(for two-sided printing)

- 1. Remove from the manual:
- Title Page
- Table of Contents pages xi and xii
- Pages 5-1 and 5-2
2. Print and insert in the manual:
- Title Page
- Table of Contents pages xi and xii
- Pages 5-1 and 5-2
- Memo to Designers (2015-01)

Note: The "AUGUST 2015" update includes the following revisions:
- Concrete mix designations and maximum aggregate size have changed to match the 2016 MnDOT Construction Specifications.
- Memo to Designers provides guidance on implementing new concrete mix design designations in bridge plans. It also provides a conversion table for old/new concrete mix design designations.

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

Beverly Farragher
Beverly Farragher
State Bridge Engineer





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Fax No: 651/366-4497

June 29, 2015

Table with 4 columns: TRANSMITTAL NOTICE (2015-02), ISSUED BY, MANUAL, DEVELOPED BY, SUBJECT. Content includes 'MnDOT Bridge Office Web Site', 'Bridge Office', 'LRFD Bridge Design Manual', and 'Title Page' through 'Section 4: Structural Analysis and Evaluation'.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This Web site should be checked regularly for updates.

INSTRUCTIONS:

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- 1. Remove from the manual:
- Title Page
- Table of Contents pages iii and iv
- Pages 3-5 and 3-6
- Pages 4-7 through 4-10
2. Print and insert in the manual:
- Title Page
- Table of Contents pages iii and iv
- Pages 3-5 and 3-6
- Pages 4-7 through 4-12

Note: The "JUNE 2015" update to Section 3 and Section 4 includes the following revisions:

- Regarding the AASHTO LRFD Article 3.6.1.3.1, added clarification that MnDOT modification to the HL-93 double truck live load applies in lieu of the load given in Article 3.6.1.3.1.
Regarding MnDOT LRFD Bridge Design Manual Article 4.6.2:
- Added reference to Bridge Preservation and Improvement Guidelines
- Added definitions for types of bridge repair projects
- Added requirements for substructures
- Included other changes meant to clarify.

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

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May 22, 2015

Table with 4 columns: TRANSMITTAL NOTICE (2015-01), ISSUED BY, MANUAL, DEVELOPED BY, SUBJECT. Content includes 'MnDOT Bridge Office Web Site', 'Bridge Office', 'LRFD Bridge Design Manual', and 'Title Page, Table of Contents, Section 3: Loads and Load Factors, Appendix A: Memos'.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This Web site should be checked regularly for updates.

INSTRUCTIONS:

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- 1. Remove from the manual:
- Title Page
- Table of Contents (iii, iv, xi, and xii)
- Section 3 (3-1 through 3-16)
- Memo to Designers 2005-01
2. Print and insert in the manual:
- Title Page
- Table of Contents (iii, iv, xi, and xii)
- Section 3 (3-1 through 3-18)

Note: The "MAY 2015" update of Section 3 includes the following major revisions:
- Removal of Memo To Designers 2005-01
- Modification to HL-93 double truck live load found in LRFD Article 3.6.1.3.1.
- Addition of MnDOT modified Strength IV. This differs from current AASHTO, but will be included in the commentary of the next AASHTO update.
- Update of fatigue limit state to match AASHTO.
- Removed MnDOT multi-presence factor variance for live load deflection check.
- Clarification of how braking force is to be applied.
- For ice loads, addition of a friction angle between ice and pier nose.
- For thermal loads on non-typical bridges, addition of guidance on bridges that include both typical and non-typical units.
- Under extreme events, additional load combination of ice and 50% of 500 year scour.
- Other changes meant to clarify.

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

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December 23, 2014

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents A. Memos		

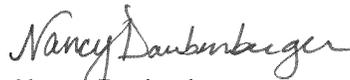
The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents (xi through xii)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (xi through xii)
  - Memo #2014-02 Inclusion of Informational Quantities in Bridge Plans (dated December 23, 2014)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or 651/366-4491.

  
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August 6, 2014

Table with 4 rows and 3 columns: DISTRIBUTION, TRANSMITTAL NOTICE, MANUAL, SUBJECT. Content includes MnDOT Bridge Office Web Site, LRFD Bridge Design Manual, and Section 5: CONCRETE STRUCTURES.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This Web site should be checked regularly for updates.

INSTRUCTIONS:

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- 1. Remove from the manual: Title Page, Table of Contents (xi through xii), Pages 5-1 & 5-2, Pages 5-107 & 5-108.
2. Print and insert in the manual: Title Page, Table of Contents (xi through xii), Pages 5-1 & 5-2, Pages 5-107 & 5-108, Memo #2014-01 AASHTO LRFD Article 5.7.3.4 Concrete Crack Control Check (dated August 6, 2014)

In the recent update dated July 31, 2014, two additional changes were missed and are included here: Low cement high performance concrete was added to the concrete mixes, Additional rebars were changed to U.S. Customary bar sizes. These changes are noted with a JULY 2014 date and a single line in the margin. This update also includes clarification on guidance for the concrete crack control check.

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

Nancy Daubenberger
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State Bridge Engineer





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July 31, 2014

Table with 4 columns: DISTRIBUTION, TRANSMITTAL NOTICE (2014-02), MANUAL, SUBJECT. It details the distribution of the LRFD Bridge Design Manual, including the title page, table of contents, and section 5 on concrete structures.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This web site should be checked regularly for updates.

INSTRUCTIONS:
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- 1. Remove from the manual:
- Title Page
- Table of Contents pages v-vi and xi-xii
- Entire Section 5 pages 5-1 through 5-136
- Memo to Designers (2011-01)
2. Print and insert in the manual:
- Title Page
- Table of Contents pages v-vi and xi-xii
- Section 5 pages 5-1 through 5-136

Note: The "JULY 2014" update of Section 5 includes the following major revisions:
- Some concrete mix designations have been changed
- Reinforcing bar sizes revised to reflect U.S. Customary bar sizes
- Diaphragm spacing requirements have been revised for p/s beam bridges
- Requirements added for p/s beam strand location
- Guidance added regarding large stool heights & horizontal shear on p/s beam bridges
- Camber multiplier for p/s beams has changed
- Guidance added on use of "MW" series beams

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual & Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

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State Bridge Engineer





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February 25, 2014

Table with 4 columns: TRANSMITTAL NOTICE (2014-01), ISSUED BY, MANUAL, DEVELOPED BY, SUBJECT. Content includes: Mn/DOT Bridge Office Web site, Bridge Office, LRFD Bridge Design Manual, Bridge Office, Title Page, Table of Contents, Section 4: STRUCTURAL ANALYSIS AND EVALUATION.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/ . This Web site should be checked regularly for updates.

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- Table of Contents (iii through iv)
- Section 4 (4-1.1 through 4-6)
2. Print and insert in the manual:
- Title Page
- Table of Contents (iii through iv)
- Section 4 (4-1 through 4-10)

Note: The "FEBRUARY 2014" update of Section 4 is the fourth update to the section. Therefore, it is being republished in its entirety. All vertical revision lines added during the previous three updates have been removed.

Major revisions include:

- Rigid cross-section equation is required for LLDF calculation for steel beam bridges
- Use of live load distribution reduction factor for moment in skewed superstructures is allowed
- Policy on design/evaluation for bridge repair projects requires use of LRFD & LRFR

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual & Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

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State Bridge Engineer





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September 13, 2013

Table with 4 columns: TRANSMITTAL NOTICE (2013-02), ISSUED BY, MANUAL, DEVELOPED BY, SUBJECT. Content includes 'Mn/DOT Bridge Office Web site', 'Bridge Office', 'LRFD Bridge Design Manual', and 'Buried Structures'.

The MnDOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/ . This Web site should be checked regularly for updates.

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- 1. Remove from the manual:
- Title Page
- Table of Contents (ix through xii)
- Section 12 (12-1 through 12-35)
2. Print and insert in the manual:
- Title Page
- Table of Contents (ix through xii)
- Section 12 (12-1 through 12-54)

Note: Due to the large number of changes, Section 12 is being republished in its entirety without any vertical revision lines in the margins. Primary changes include updating the box culvert design guidance and example.

Any technical questions regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual & Policy Engineer, at dave.dahlberg@state.mn.us or 651/366-4491.

Nancy Daubenberger (handwritten signature)

Nancy Daubenberger
State Bridge Engineer





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April 17, 2013

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Title Page Table of Contents A. Memos		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents (xi through xii)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (xi through xii)
  - Memo #2013-01 Conversion from Metric to U.S. Customary Rebar Designations (dated April 17, 2013)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or by phone at 651/366-4491.

Nancy Daubenberger  
State Bridge Engineer





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November 21, 2012

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<b>SUBJECT</b>	Title Page Table of Contents A. Memos		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents (xi through xii)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (xi through xii)
  - Memo #2012-02 Transition to New MnDOT Pile Formula 2012 (MPF12) (dated November 21, 2012)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or by phone at 651/366-4491.

Nancy Daubenberger  
State Bridge Engineer





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April 12, 2012

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<b>SUBJECT</b>	Title Page Table of Contents A. Memos		

The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents (xi through xii)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (xi through xii)
  - Memo #2012-01 Discontinued Usage of Plain Elastomeric Bearing Pads and Substitution with Cotton-Duck Bearing Pads (dated April 12, 2012)

Any *technical questions* regarding this transmittal should be directed to Dave Dahlberg, Bridge Design Manual and Policy Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or by phone at 651/366-4491.

  
Nancy Daubenberger  
FOR State Bridge Engineer





**Minnesota Department of Transportation**

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Fax No: 651/366-4497

December 23, 2011

<b>DISTRIBUTION:</b> MnDOT Bridge Office Web Site			
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<b>SUBJECT</b>	Title Page Table of Contents A. Memos		

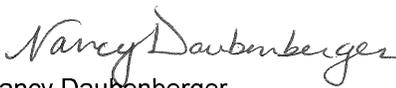
The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in Adobe PDF (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

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1. Remove from the manual:
  - Title Page
  - Table of Contents (xi through xii)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (xi through xii)
  - Memo #2011-03 Interim Guidance for Installation of Temporary Barriers on Bridges and Approach Panels (dated December 23, 2011)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or by phone at 651/366-4491.

  
Nancy Daubenberger  
State Bridge Engineer





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Fax No: 651/366-4497

October 5, 2011

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<b>SUBJECT</b>	Title Page Table of Contents Section 15: BRIDGE LOAD RATING		

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1. Remove from the manual:
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  - Table of Contents (ix through xii)
  - Section 15 (15-9 through 15-14, 15-17 through 15-20, and 15-23 through 15-26)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (ix through xii)
  - Section 15 (15-9.1 through 15-14, 15-17 through 15-20, and 15-23 through 15-26.3)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Engineer, at [dave.dahlberg@state.mn.us](mailto:dave.dahlberg@state.mn.us) or by phone at 651/366-4491.

  
Nancy Daubenberger  
State Bridge Engineer





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September 14, 2011

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<b>SUBJECT</b>	Title Page Table of Contents Section 2: GENERAL DESIGN AND LOCATION FEATURES		

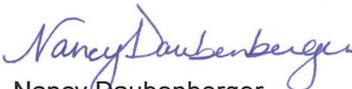
The MnDOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

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1. Remove from the manual:
  - Title Page
  - Table of Contents (i through viii and xi through xii)
  - Section 2 (2-49 through 2-52)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (i through viii and xi through xii)
  - Section 2 (2-49 through 2-52)

Any **technical questions** regarding this transmittal should be directed to Arielle Ehrlich, Acting LRFD Engineer, at [arielle.ehrlich@state.mn.us](mailto:arielle.ehrlich@state.mn.us) or by phone at 651/366-4515.

  
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June 30, 2011

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  - Section 4 (4-1 and 4-2)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (iii and iv)
  - Section 4 (4-1.1 through 4-2)

Any **technical questions** regarding this transmittal should be directed to Arielle Ehrlich, Acting LRFD Engineer, at [arielle.ehrlich@state.mn.us](mailto:arielle.ehrlich@state.mn.us) or by phone at 651/366-4515.

  
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December 1, 2010

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The Mn/DOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/>. This Web site should be checked regularly for updates.

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  - Section 1 (1-1 through 1-21)
  
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  - Table of Contents (i and ii)
  - Section 1 (1-1 through 1-26)

Any **technical questions** regarding this transmittal should be directed to Arielle Ehrlich, Acting LRFD Engineer, at [arielle.ehrlich@state.mn.us](mailto:arielle.ehrlich@state.mn.us) or by phone at 651/366-4515.

Duane R. Hill  
Acting State Bridge Engineer





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June 30, 2010

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1. Remove from the manual:
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  - Section 11 (11-1 through 11-20)
2. Print and insert in the manual:
  - Title Page
  - Table of Contents (vii and viii)
  - Section 11 (11-1 through 11-20)

Any **technical questions** regarding this transmittal should be directed to Arielle Ehrlich, Acting LRFD Engineer, at [arielle.ehrlich@state.mn.us](mailto:arielle.ehrlich@state.mn.us) or by phone at 651/366-4515.

*Nancy Daubenberger*  
for Duane R. Hill  
Acting State Bridge Engineer





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March 30, 2010

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<b>SUBJECT</b>	Title Page Table of Contents Section 9: DECKS AND DECK SYSTEMS		

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Any **technical questions** regarding this transmittal should be directed to David Dahlberg, LRFD Engineer, at [dave.dahlberg@dot.state.mn.us](mailto:dave.dahlberg@dot.state.mn.us) or by phone at 651/366-4491.

FOR Daniel L. Dorgan  
State Bridge Engineer





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  - Table of Contents (v through vi)
  - Section 5 (5-1 through 5-136)

Note: Due to the large number of changes, Section 5 is being republished in its entirety without any vertical revision lines in the margins. The main text, the slab span example, and the prestressed beam example have been updated. The post-tensioned slab example has not been updated yet.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Engineer, at 651/366-4491.

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  - 4-3 through 4-8
  - 6-3 through 6-10
  
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  - Table of Contents (iii and iv)
  - 4-3 through 4-6
  - 6-3 through 6-10

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Any **technical questions** regarding this transmittal should be directed to David Dahlberg, LRFD Engineer, at [dave.dahlberg@dot.state.mn.us](mailto:dave.dahlberg@dot.state.mn.us) or by phone at 651/366-4491.

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  - MEMO #2008-02 Truss Bridge Gusset Plate Analysis (dated October 20, 2008)

Any **technical questions** regarding this transmittal should be directed to David Dahlberg, LRFD Engineer, at [dave.dahlberg@dot.state.mn.us](mailto:dave.dahlberg@dot.state.mn.us) or by phone at 651/366-4491.

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  - MEMO #2008-01 Prestressed Concrete Design – Calculation of Prestress Losses and Beam Camber & Deflection (dated September 18, 2008)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/366-4491.

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Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Engineer, at 651/366-4491.

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  - Section 6 (6-1 through 6-124)

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Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Engineer, at 651/366-4491.

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Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/366-4491.

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Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/366-4491.

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  - Manual Cover (ii)
  - Table of Contents (ix and x)
  - MEMO #2007-02 Adhesive Anchors Under Sustained Tensile Loads (dated October 3, 2007)
  - MEMO #2007-03 Prestressed Beam Camber and Deflection (dated October 3, 2007)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/366-4491.

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  - MEMO #2007-01 Bridge Office Substructure Protection Policy (dated July 23, 2007)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/366-4491.

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Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/366-4491.

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  - 2-1 through 2-116 (entire section)

Note: The "FEBRUARY 2007" update of Section 2 is the fourth update to the section. Therefore, it is being republished in its entirety. All vertical revision lines added during the previous three updates have been removed. A single vertical line in the margin indicates a revision made to the text during the fourth update.

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/747-2116.

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  - 4-1 through 4-8 (entire section)

Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651/747-2116.

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Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

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  - 5-1 and 5-2, 5-29 and 5-30
  - 13-1 through 13-41 (entire section)
  - MEMO #2006-01 New MN63 Prestressed Concrete Beam (dated May 31, 2006)
3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

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April 5, 2006

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3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

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3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT</b>	Manual Cover Table of Contents Section 5: CONCRETE STRUCTURES A. MEMOS		

The Mn/DOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/> . This Web site should be checked regularly for updates.

The following pages in Section 5 have been changed:  
5-2, 5-17, 5-19, 5-20, 5-22, 5-25, 5-28, 5-29, 5-30, 5-36, 5-43, 5-44, 5-77, 5-82, 5-99, 5-119, 5-120 and 5-121.

**INSTRUCTIONS:**  
(for two-sided printing)

1. Remove from the manual:
  - Manual Cover (ii)
  - Table of Contents (ix and x)
  - 5-1 and 5-2, 5-17 through 5-22, 5-25 through 5-30, 5-35 and 5-36, 5-43 and 5-44, 5-77 and 5-78, 5-81 and 5-82, 5-99 and 5-100, 5-119 through 5-122.
2. Print and insert in the manual:
  - Manual Cover (ii)
  - Table of Contents (ix and x)
  - 5-1 and 5-2, 5-17 through 5-22, 5-25 through 5-30, 5-35 and 5-36, 5-43 and 5-44, 5-77 and 5-78, 5-81 and 5-82, 5-99 and 5-100, 5-119 through 5-122.
  - MEMO #2005-02 Discontinue Use of Cast Bearing Option (dated November 9, 2005)
  - MEMO #2005-03 New MN45 and MN54 Prestressed Concrete Beams (dated November 10, 2005)
3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

Daniel L. Dorgan  
State Bridge Engineer





Minnesota Department of Transportation

Bridge Office
Mail Stop 610
3485 Hadley Avenue North
Oakdale, MN 55128-3307

Phone No: (651) 747-2100
Fax No: (651) 747-2108

March 24, 2005

Table with 4 rows and 3 columns. Row 1: DISTRIBUTION: Mn/DOT Bridge Office Web site. Row 2: TRANSMITTAL NOTICE (2005-02), ISSUED BY, Bridge Office. Row 3: MANUAL, LRFD Bridge Design Manual, DEVELOPED BY, Bridge Office. Row 4: SUBJECT, Manual Cover, Section 2: GENERAL DESIGN AND LOCATION FEATURES.

The Mn/DOT Bridge Office LRFD Bridge Design Manual is available for download in Adobe PDF (Portable Document Format) at http://www.dot.state.mn.us/bridge/. This Web site should be checked regularly for updates.

The following pages in Section 2 have been changed:

2-4, 2-5, 2-6, 2-7, 2-8, 2-9, 2-10, 2-11, 2-12, 2-13, 2-14, 2-15, 2-19, 2-21, 2-22, 2-30, 2-31, 2-41, 2-42, 2-53, 2-62, 2-88, 2-89, 2-90, 2-94, 2-95, 2-96, 2-97, 2-100, 2-101, 2-102, 2-103, 2-104, 2-105, 2-106, 2-107, 2-108, 2-109, 2-111, 2-112, 2-113, and 2-114.

INSTRUCTIONS:

(for two-sided printing)

- 1. Remove from the manual:
- Manual Cover (ii)
- 2-3 through 2-16, 2-19 through 2-22, 2-29 through 2-32, 2-41 and 2-42, 2-53 and 2-54, 2-61 and 2-62, 2-87 through 2-90, 2-93 through 2-114.
2. Print and insert in the manual:
- Manual Cover (ii)
- 2-3 through 2-16, 2-19 through 2-22, 2-29 through 2-32, 2-41 and 2-42, 2-53 and 2-54, 2-61 and 2-62, 2-87 through 2-90, 2-93 through 2-114.
3. Any technical questions regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

Handwritten signature of Daniel L. Dorgan

Daniel L. Dorgan
State Bridge Engineer





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March 22, 2005

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<b>SUBJECT:</b>	Table of Contents A. MEMOS		

The Mn/DOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/> . This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

1. Remove from the manual:
  - Table of Contents (ix and x)
2. Print and insert in the manual:
  - Table of Contents (ix and x)
  - Memo #2005-01 LRFD and Bridge Load Rating Issues (dated 02-14-2005)

You may already have this memo.  
Replace it with the one attached to this transmittal notice,  
which is numbered in the lower-right corner.

3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

Daniel L. Dorgan  
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December 17, 2004

<b>DISTRIBUTION:</b> Mn/DOT Bridge Office Web site			
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<b>SUBJECT:</b>	Manual Cover and Table of Contents Section 9: DECKS AND DECK SYSTEMS		

The Mn/DOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/> . This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

1. Remove from the manual:
  - Manual Cover (ii) and Table of Contents (vii and viii)
  - Section 9 (9-1 through 9-30, inclusive)
2. Print and insert in the manual:
  - Manual Cover (ii) and Table of Contents (vii and viii)
  - Section 9 (9-1 through 9-34, inclusive)
3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

Daniel L. Dorgan  
State Bridge Engineer





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October 20, 2004

<b>DISTRIBUTION:</b> Mn/DOT Bridge Office Web site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT:</b>	Manual Cover and Table of Contents Section 5: CONCRETE STRUCTURES		

The Mn/DOT Bridge Office *LRFD Bridge Design Manual* is available for download in Adobe PDF (Portable Document Format) at <http://www.dot.state.mn.us/bridge/> . This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

1. Remove from the manual:
  - Manual Cover (ii) and Table of Contents (i through x, inclusive)
  - Section 5 (5-1 through 5-124, inclusive)
2. Print and insert in the manual:
  - Manual Cover (ii) and Table of Contents (i through x, inclusive)
  - Section 5 (5-1 through 5-140, inclusive)
3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at (651) 747-2116.

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April 30, 2004

<b>DISTRIBUTION:</b> Mn/DOT Bridge Office Web site			
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<b>SUBJECT:</b> Section 2: GENERAL DESIGN AND LOCATION FEATURES			

The Mn/DOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/> . This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

1. Remove from the manual: Section 2, pages 2-1 through 2-121 inclusive.
2. Print and insert in the manual: Section 2, pages 2-1 through 2-116 inclusive.
3. Any **technical questions** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651-747-2116.

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April 30, 2004

<b>DISTRIBUTION:</b> Mn/DOT Bridge Office Web site			
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<b>MANUAL</b>	LRFD Bridge Design Manual	<b>DEVELOPED BY</b>	Bridge Office
<b>SUBJECT:</b> Manual Cover and Table of Contents			

The Mn/DOT Bridge Office *LRFD Bridge Design Manual* is available for download in *Adobe PDF* (Portable Document Format) at <http://www.dot.state.mn.us/bridge/> . This Web site should be checked regularly for updates.

**INSTRUCTIONS:**

1. Remove from the manual: Manual Cover (page ii) and Table of Contents (page i, ii, and iii).
2. Print and insert in the manual: Manual Cover (page ii) and Table of Contents (page i, ii, and iii).
3. Any ***technical questions*** regarding this transmittal should be directed to Dave Dahlberg, LRFD Implementation, at 651-747-2116.

A handwritten signature in blue ink that reads "Daniel L. Dorgan".

Daniel L. Dorgan  
State Bridge Engineer

