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2.1 Organizational Charts

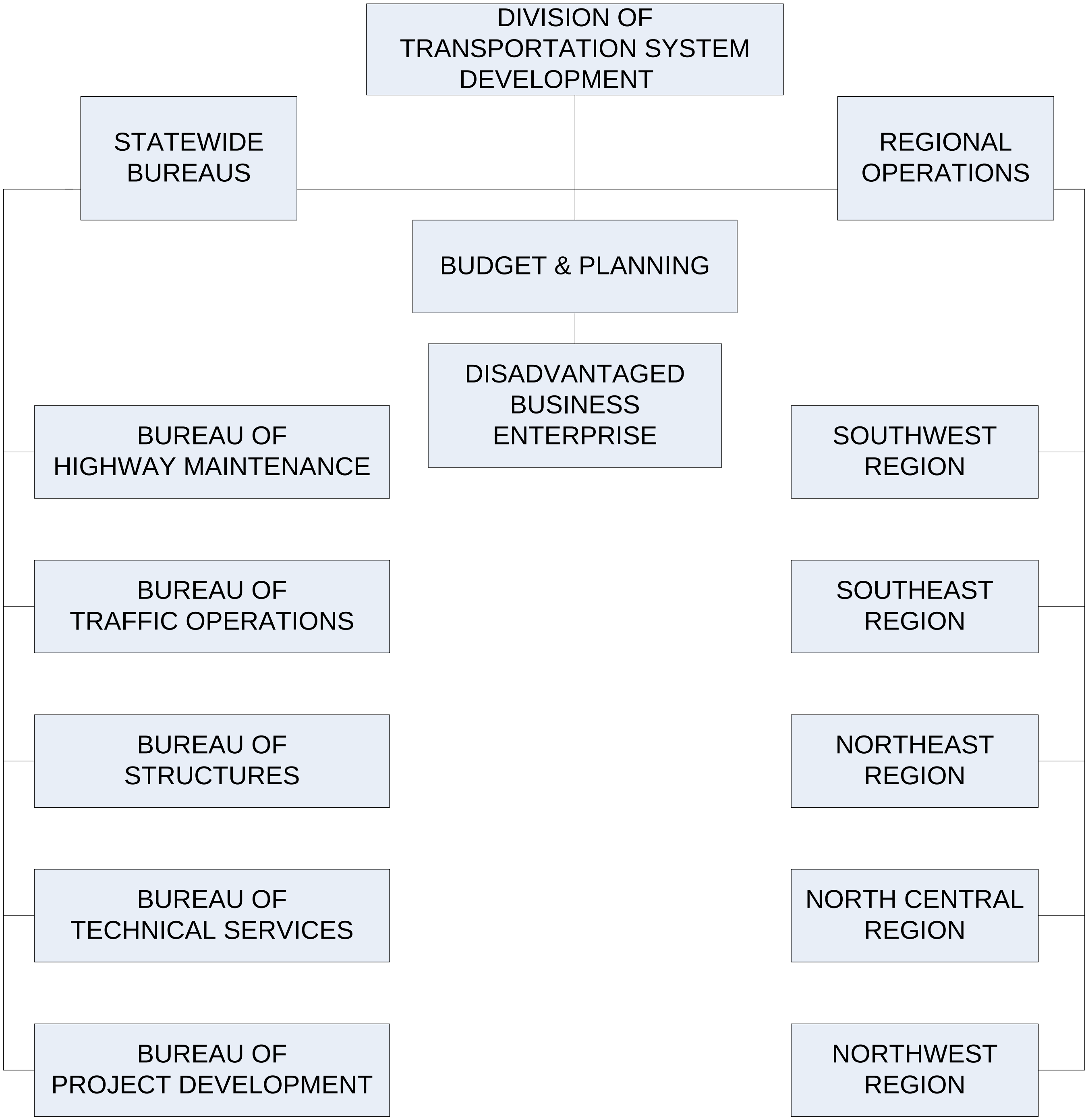

Figure 2.1-1
Division of Transportation System Development

f. Select at least one other structure person to go to the bridge site.

5. Observe all safety rules at bridge site.

6. Continue to communicate with all Bureau staff.

   a. Select at least one other structure person to go to Notify Bureau Contact Person to perform required communication.

7. Document actions taken and file for future reference. Communicate to all indicated in Item #2.

2.2.4 Public Communication Record

A "Public Communication Record": (PCR) is a form filled out by DOT employees to inform upper management and other potentially interested staff of a contact that may be of interest to the recipient. The contact is normally from the Media, Legislator, Local Official or the public concerning a topic that is or could be controversial now or in the future.

Within the Bureau of Structures (BOS), the Bureau Director, Section Managers, Supervisors and Lead workers should be included in all PCRs filled out by BOS staff, along with the established list of PCR contacts found in Outlook "Global Address List" under DOT DL PCR. A copy of the PCR form (DT 33) can be found on the DOTNET at:

http://dotnet/opa/opapolicies.htm

If you are contacted by the Media, Legislator or Local Official and are not sure if you need to fill out a PCR, contact your Supervisor for their opinion. A PCR is quick and easy to do so "if in doubt fill it out" is the best approach to use.

2.3 Responsibilities of Bureau of Structures

2.3.1 Structures Design Section

- Provide guidance to Regional Offices on the preparation of various types of Structure Survey Reports.

- Assist Regional Offices making design investigation studies by providing guidance on structure costs, depths, and practical structure types for the alternate sites under consideration.

- Prepare comparative cost estimates for alternate structure types. Prepare economic studies on rehabilitation versus replacement of existing structure. Make recommendations to Regional Office or Consultant or Government Agency.

- Review and approve Consultant preliminary and final plans, evaluate hydraulic adequacy and compliance to current Standards.

- Review and approve Consultant rehabilitation proposals.

- Collect and make information available to Regional Offices for hydrology studies and new hydraulic developments by other agencies.

- Provide procedures for scour analysis of structures.

- Make field observations of the proposed site, gather additional information for hydraulic reports, and evaluate the general conditions of the site. Coordinate hydraulic impacts with DNR.

- Assemble data and prepare drawings as required by Coast Guard for permit applications to construct bridges over navigable streams. Assemble data as necessary and receive certification from the Corps of Engineers and other agencies exercising environmental control over the proposed structure improvement.

- Prepare preliminary structures plans for bridges. This includes designing, detailing, drafting, estimating, and checking as may be necessary to obtain approvals from other governmental agencies.

- Determine size and length of box culverts. Design and plot culvert plans for checking by staff.

- Distribute preliminary structure plans to Regional Offices for approval and utility contacts.

- Prepare final contract plans for bridges, box culverts and other structures which include designing, detailing, drafting, estimating and ensuring compliance with preliminary study report and Standard Specifications.

2.4 Bridge Standards and Insert Sheets

Bridge standards are drawings which show the standard practice for details used by WisDOT. These Standards have been developed over time by input from individuals involved in design, construction and maintenance. They are applicable to most structures and should be used unless exceptions are approved by the Section Managers.

The Insert Sheets represent the Standards and are intended to be used with minimum revision for insertion in the final set of plans for construction purposes.

1. FHWA Approval of Structure Standards Process

   The following points define the working relationship between FHWA and WISDOT concerning production and adoption of Bureau of Structures (BOS) Standard Detail Drawings. These points were agreed upon at a meeting on December 17, 2002 between BOS and FHWA.

- Submittals will be sent by electronic methods in PDF format to FHWA. (For special cases with a large amount of supporting information other methods may be used as agreed to by both parties on a case by case basis).

- Generally two weeks should be sufficient to render an approval or request for additional information. (In special cases requiring input from sources outside of the Wisconsin FHWA office additional time will be requested in writing with an expected due date for a decision agreed to by both WisDOT and FHWA).

- Appropriate supporting documentation ranging from written explanations to fully detailed engineering calculations will accompany submittals. The level of support should reflect the level of review expected.

- The Structure Standards reviewed by the FHWA will be done so with respect to Federal Law, Policy and safety issues. Differing opinions on other issues will not be cause for non-approval of standards.

2.5 Structure Numbers

An official number referred to as the Structure Number is assigned to every structure on the State Highway System for the purpose of having a definite designation. The Structure Number is hyphenated with the first letter being either a B, C, P, S, R, M or N. B is assigned to all structures over 20 ft. in length, including culvert configurations. In general, C is assigned to structures 20 ft. or less in length with an exception being that box culverts must have a cross sectional area greater than, or equal to 20 square feet to be assigned a C number. Do not include pipes unless they meet the definition of a bridge. A set of nested pipes may be given a Bridge Number if the distance between the outside walls of the end pipes exceeds 20 ft. and the clear distance between pipe openings is less than half the diameter of the smallest pipe. P designates structures for which there are no structural plans on file. S is for sign structures, R is for Retaining Walls, and N is for Noise Barriers. M is for miscellaneous structures where it is desirable to have a plan record. Bridges on state boundary lines also have a number designated by the adjacent state.

WisDOT Policy Item:

No new P numbers will be assigned as we should always request plans.

Regional Offices should assign numbers to structures before submitting information to the Bureau of Structures for the structural design process or the plan review process. Unit numbers are only assigned to long bridges or complex interchanges where it is desirable to have only one bridge number for the site.

For guidance on inspection and documentation of state-owned small bridge structures (C-XX-XXX), see the Structures Inspection Manual.

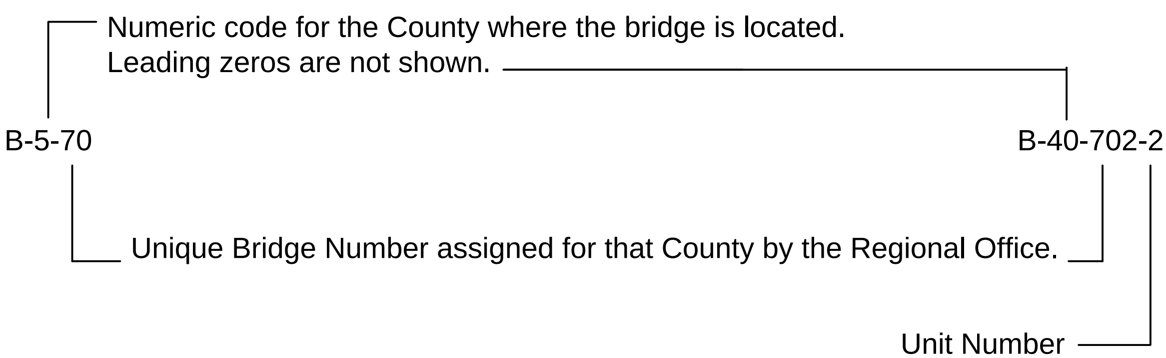

Figure 2.5-1
Bridge Number Detail

See 14.1.1.1 for criteria as to when a retaining wall gets assigned a R number and receives a name plate. A Structure Survey Report should be sent to the Structures Design Section, even if designed by the Regional Office.

See Section 6.3.3.7 for guidance on location of name plate on structures.

When a structure is rehabilitated, the name plate should be preserved, if possible, and reinstalled on the rehabilitated Structure. If a new name plate is required, it should show the year of original construction. The original structure number applies to all rehabilitation including widening, lengthening, superstructure replacement, etc.

Pedestrian only bridges get a B number if they are state maintained or cross a roadway. Otherwise use an M number for tracking purposes such as DNR bridges reviewed by DOT.

2.6 Bridge Files

Records and information useful in bridge planning and design are kept in appropriate places. Following is a brief summary of the various types of files, their contents and location. The data is arranged in alphabetical order for quick reference.`

		Location	Agency
Bridge Cost Analysis		Structures Design	BOS
National Bridge Inventory Data			
	Information coded for the electronic computer file.	Structures Development	BOS
Catalogues		Structures Development	BOS
	Manufacturers' Product Files,		
	Research Files and Technical Items,		
	Civil, Mechanical, and Electrical,		
	Technical Reference Books		
Design Calculations			
	After project is completed, the design calculations are filed in a folder until they are digitized.	Bridge Files, Microfilm or in HSIS	BOS
Engineers' Estimates		-----------	BPD
FHWA Program Manual		-----------	BOS
Log of Test Borings		Geotechnical Section	BTS
	Records of all borings.		
	Borings for each bridge are kept in Bridge Folder or on microfilm.		
Manuals		Structures Development	BOS
	Bridge Manual, Computer,		
	Construction and Materials Manual,		
	Design Manual, Maintenance Manual,		
	Transportation Administrative Manual		
Maps		Structures Design	BOS
	Geological Maps, National Forests,		
	Navigation Charts, Rivers-Harbors,		
	State Park, Topographic, Historical		

2.8 Special Provisions

Special provisions are required for some projects to give special directions or requirements that are not otherwise satisfactorily detailed or prescribed in the standard specifications. Following are some of the principal functions of the special provisions:

1. Supplement the Standard Specifications by setting forth requirements which are not adequately covered, for the proposed project, by the Standard Specifications.

2. Alter the requirements of the Standard Specifications where such requirements are not appropriate for the proposed work.

3. Supplement the plans with verbal requirements where such requirements are too lengthy to be shown on the plans.

4. Call the bidder's attention to any unusual conditions, regulations or laws affecting the work.

5. For experimental use of a new material or system such as paint systems not covered in the Standard Specifications.

When preparing the special provisions for any project, the writer must visualize the project from the standpoint of the problems that may occur during construction.

Special provisions are generally written for a specific project or structure, however several “standard” bridge special provisions are available on-line at the Structures Design Information site:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/special-provisions.aspx

These special provisions may require modification to accurately reflect the requirements of individual projects or structures.

2.9 Terminology

AASHTO	American Association of State Highway and Transportation Officials.
ABUTMENT	Supports at the end of the bridge used to retain the approach embankment and carry the vertical and horizontal loads from the superstructure.
ACI	American Concrete Institute.
AISC	American Institute of Steel Construction.
Allowable Headwater	The maximum elevation to which water may be ponded upstream of a culvert or structure as specified by law or design.
Anchor Bolts	Bolts that are embedded in concrete which are used to attach an object to the concrete such as rail posts, bearings, etc.
ANSI	American National Standards Institute.
Apron	The paved area between wingwalls at the end of a culvert.
ASTM	American Society for Testing Materials.
ADT	Average Daily Traffic
Award	The decision to accept the proposal of the lowest responsible bidder for the specified work, subject to the execution and approval of a satisfactory contract bond and other conditions as may be specified or required by law.
AWS	American Welding Society.
Backfill	Fill materials placed between structural elements and existing embankment.
Backwater	An unnaturally high stage in a stream caused by obstruction of flow, as by a dam, a levee, or a bridge opening. Its measure is the excess of unnatural over natural stage. A back up of water due to a restriction.
Bar Chair	A device used to support horizontal reinforcing bars above the base of the form before the concrete is poured.
Bar Cutting Diagram	A diagram used in the detailing of bar steel reinforcement where the bar lengths vary as a straight line.
Base Course	The layer of specified material of designed thickness placed on a subbase or a subgrade to support a surface course.
Batter Pile	A pile that is purposely driven at an angle with vertical.
Bearings	Device to transfer girder reactions without overstressing the supports, insuring the bridge functions as intended. (See Fixed Bearings and Expansion Bearings).
Bearing Stiffener	A stiffener used at points of support on a steel beam to transmit the load from the top of the beam to the support point.
Bedrock	The solid rock underlying soils or other superficial formation.
Bench Mark	A relatively permanent object bearing a marked point whose elevation above or below an adopted datum is known.
Blocking Diagram	A diagram which shows the distance from a horizontal line to all significant points on a girder as it will be during erection.
Bridge	A structure having a span of more than 20 ft. from face to face of abutments, measured along the roadway centerline.

2.10 WisDOT Bridge History

Prior to the early 1950's, structure types on Wisconsin State Highways were predominantly reinforced concrete slabs and steel girders or trusses with reinforced concrete decks. Also, timber structures were used at a number of county and town road sites. In 1952, the first prestressed concrete voided slab sections were cast and erected incorporating transverse post-tensioning. In 1956, the first prestressed concrete "I" girders were designed and precast. After field setting, these prestressed girders were post-tensioned and completed with an integral cast-in-place reinforced concrete deck. During the mid 1950's and early 1960's, prestressed concrete "I" and steel girder structures were made continuous and incorporated composite designs for carrying live loads.

In 1971, the first cable-stayed bridge in the United States, a three span pedestrian structure, was constructed in Menomonee Falls.

2.10.1 Unique Structures

Structure Type	Bridge Number	Year Constructed	(feet) Span Configuration
Steel Rigid Frames	B-40-48-Milwaukee	1959	45.3, 168.5, 46.3
Steel Rigid Frames	B-56-47/48*-Mirror Lake	1961	50.6, 22-.0, 49.4
Overhead Timber Truss	B-22-50*-Cassville	1962	48.0
Arch Truss	B-16-5-Superior	1961	270.0, 600.1, 270.0
Tied Arches	B-9-87*-Cornell	1971	485.0
Tied Arches	B-12-27*-Prairie du Chien	1974	462.0
Tied Arches	B-40-400-Milwaukee	1974	270.0, 600, 270.0
Tied Arches	B-5-158*-Green Bay	1980	450.1
Tied Arches	B-22-60-Dubuque, IA	1982	670.0
Tied Arches	B-16-38*-Superior	1984	500.0
Prestressed “I” Girders with Cantilever	B-40-524*-Milwaukee	1985	112.0, 69.0, 107.8, 383.5 Spans with 25’ Cantilevers
Prestressed Strutted Arches	B-40-603-Milwaukee	1992	8-158.0 Strutted Arch Spans
Tied Arches	B-32-202* - LaCrosse	2004	475’

Table 2.10-1
Unique Structures

* Designed in the Wisconsin Department of Transportation Bureau of Structures.

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4.1 Introduction

Transportation structures, such as bridges and retaining walls, have a strong influence on the appearance of transportation projects, as well as the overall appearance of the general vicinity of the project. In locations where there is an opportunity to appreciate such structures, it is often desirable to add aesthetic enhancements to fit the project site.

Desirable bridge aesthetics do not necessarily need to cost much, if any, additional money. Aesthetic enhancements can be made in a number of ways. Primary features such as structure type and shape have the most influence on appearance, with color and texture playing secondary roles. Formliners, especially when used in conjunction with a multi-colored stain, are more expensive than one or two single color stains on smooth concrete, and have on a number of occasions not fit the context of the project. It is the responsibility of the design team to identify aesthetic treatments that are consistent with the environment and goals of the project, are maintainable over the life of the structures, and are cost effective.

While initial cost for aesthetic enhancements is a concern, it has become apparent that maintenance costs can be considerably more than initial costs. Stain, which acts more like paint, must be periodically redone. Such reapplication oftentimes requires lane closures which are both an undesirable inconvenience to users and come with a significant cost associated with maintenance-of-traffic.

4.5 Aesthetics Process

A number of parties can be responsible for the appearance of a structure, as well as the project as a whole. The structural design engineer should be instrumental in leading the aesthetic design process, a process that may include the Region, the Bureau of Structures, the public and aesthetic advisors (architects, landscape architects, urban planners, artists, etc).

Public input comes in a variety of ways. Advisory groups, special interest groups and general public information meetings are all ways to receive public input and are part of the CSS (Community Sensitive Solutions) process.

The structural design engineer needs to be involved early in the aesthetic decision making process. BOS should have early representation on projects with considerable aesthetic concerns.

WisDOT policy item:

The 2015-2017 budget bill reduced State CSS funding to zero. Very low cost aesthetic enhancements through appropriate shape and geometric relief are allowed. See 4.3 for discussion on primary features such as shape. Geometric relief is defined as:

- Rustications produced by cut (likely) wood (e.g. rustication lines)
- Formliners such as ribbed or broken ribbed
- Formliners that do not replicate other objects (e.g. rocks or cut stone)
- Shapes that do not depict anything pictorially (e.g. animals, flowers, sailboats, etc.)

Items considered CSS (not state or federal funded on state and local projects that are eligible for state funding)*:

- Stain**
- Formliner, other than the geometric formliner defined above**
- Pedestrian railing or fencing other than that shown in the WisDOT Bridge Manual Standards (Maintenance of *all* fence and/or railing coatings, other than galvanization, is the responsibility of the Municipality and should be covered in the SMA***.)
- Ornamentation, including city symbols, street names, etc.
- Non-standard lighting and sign supports
- Structure shapes that are not as defined in 4.3

* CSS items also require a State-Municipal Agreement (SMA)*** that makes local municipalities responsible for future maintenance and all associated costs.

** At the time the 2015-2017 budget was passed, formliner and/or staining were not considered CSS items, making them eligible for standard improvement project funds.

For lettings before **August 15, 2016** formliner and/or stain will be classified as eligible improvement items. A signed SMA*** is required and should state the Municipality will be responsible for maintaining the concrete surface and staining.

For lettings after **August 15, 2016** all state projects, and any local projects eligible for state funds, that include formliner and/or staining must have an SMA*** signed and dated prior to August 15, 2016 stating that the cost for formliner and/or staining will be paid for by project funds (state and/or federal); otherwise the items must be funded with 100% local funds. The SMA shall also state that the Municipality will be responsible for maintaining the concrete surface and staining. Exceptions to this policy include projects where the aesthetic items are required in order to be eligible for federal funds (e.g. NEPA documents, federal program requirements). In these cases the items would be eligible for state and federal funds, and long-term maintenance will be the responsibility of the facility owner. Local projects that are not eligible for state funds are not impacted by this policy.

*** SMA, or a maintenance agreement between the local municipality and the state, or another agreement determined by the state.

4.6 Levels of Aesthetics

The Regional Office should establish one of the following levels of aesthetics and indicate it on the Structure Survey Report. This will help the structural designer decide what level of effort and possible types of aesthetics treatments to consider. If Level 2 or greater is indicated, the Regional Office personnel or consultant must suggest particular requirements such as railing type, pier shape, special form liners, color, etc. in the comments area of the Structure Survey Report. Most Regions/municipalities prefer to leave anti-graffiti coating off of structures and would rather re-stain, as this is easier than trying to clean the graffiti.

Aesthetic treatments should be agreed upon prior to completion of preliminary plans in order for the final design to proceed efficiently. These details would be developed through the aesthetic process.

1. Level One: A general structure designed with standard structure details. This would apply in rural areas and urban areas with industrial development.

2. Level Two: Consists of cosmetic improvements to conventional Department structure types, such as the use of color stains/paints, texturing surfaces, modifications to fascia walls and beams or more pleasing shapes for columns. This would apply where there needs to be less visual impact from roadway structures.

3. Level Three: Emphasize full integration of efficiency, economy and elegance in structure components and the structure as a whole. Consider structure systems that are pleasing such as shaped piers and smooth superstructure lines. These structures would need to be in harmony with the surrounding buildings and/or the existing landscape.

4. Level Four: Provide overall aesthetics at the site with the structure incorporating level three requirements. The structure would need to blend with the surrounding terrain and landscaping treatment would be required to complete the appearance.

Note: The above text was left in this chapter, but will likely be modified or removed in future editions of this Manual. See 4.9 for current policy regarding CSS and levels of aesthetics.

4.7 Accent Lighting for Significant Bridges

The Wisconsin DOT will consider as part of an improvement project accent lighting for significant urban bridges with a clear span length of 450 feet and greater. The lighting would accent significant components above the driving surface such as an arch, truss, or a cable stayed superstructure. This lighting would enhance the noteworthy structure components of these significant bridges. The Traffic Guideline Manual (TGM) and the Highway Program Manual (HPM) have respective guidance of maintenance and cost share policy.

The following structures would fall into this definition of significant urban bridges:

"Name"	Region	County	Feature On	Feature Under	Year Built	Border
Tower Drive	NE	Brown	IH 43	Fox River	1979	
Praire du Chien	SW	Crawford	USH 18-STH 60	Mississippi River	1974	X
Blatnik	NW	Douglas	IH 535-USH 53	St Louis Bay	1961	X
Bong	NW	Douglas	USH 2	St Louis River	1983	X
Cass Arch	SW	La Crosse	USH 14 EB	Mississippi River	2004	X
Cass Truss	SW	La Crosse	USH 14 WB	Mississippi River	1940	X
Hoan Bridge	SE	Milwaukee	IH 794 WB-Lake Freeway	Milwaukee River	1974	
Dubuque (Iowa)	SW	Grant	USH 61-USH 151	Mississippi River	1982	X
Stillwater	NW	St Croix	TH 36	St Croix River	New	X

Table 4.4-1
Accent Lighting for Significant Bridges

4.8 Resources on Aesthetics

The Bridge Aesthetic Sourcebook from AASHTO is a very good source of practical ideas for short and medium span bridges. The Transportation Research Board (TRB) Subcommittee on Bridge Aesthetics authored this document and it can be found on the following website: The final printing of this guide (noted in the References) is available through the AASHTO publication website:

4.9 Non-CSS Aesthetic Concepts

Standards 4.02-4.05 provide details for acceptable non-CSS funded aesthetic concepts. The three types (Type I, Type II and Type III) show a plain wing, a wing with a rustication trim line and a wing with a recessed panel, respectively. For each given wing type, one or two acceptable parapet and/or pier details are shown.

- Type I: Simple features utilizing a plain wing, standard parapet and minimal pier rustications. Type I is ideal for most rural and some urban applications.

- Type II: The wings utilize the same rustication trim line as the columns. The columns can have single or paired rustication trim lines. Single rustication lines can be used for 32-inch parapets and double rustication lines can be used for 42-inch parapets. Type II can be used in urban applications and other limited areas.

- Type III: Recessed panel wings and recessed panel columns, along with standard parapets, are to be used in urban settings, only.

Within a given corridor, only one Type should be chosen so as not to create a disharmonious experience for those driving the corridor.

The following pages show renderings of the various non-CSS aesthetic concepts.

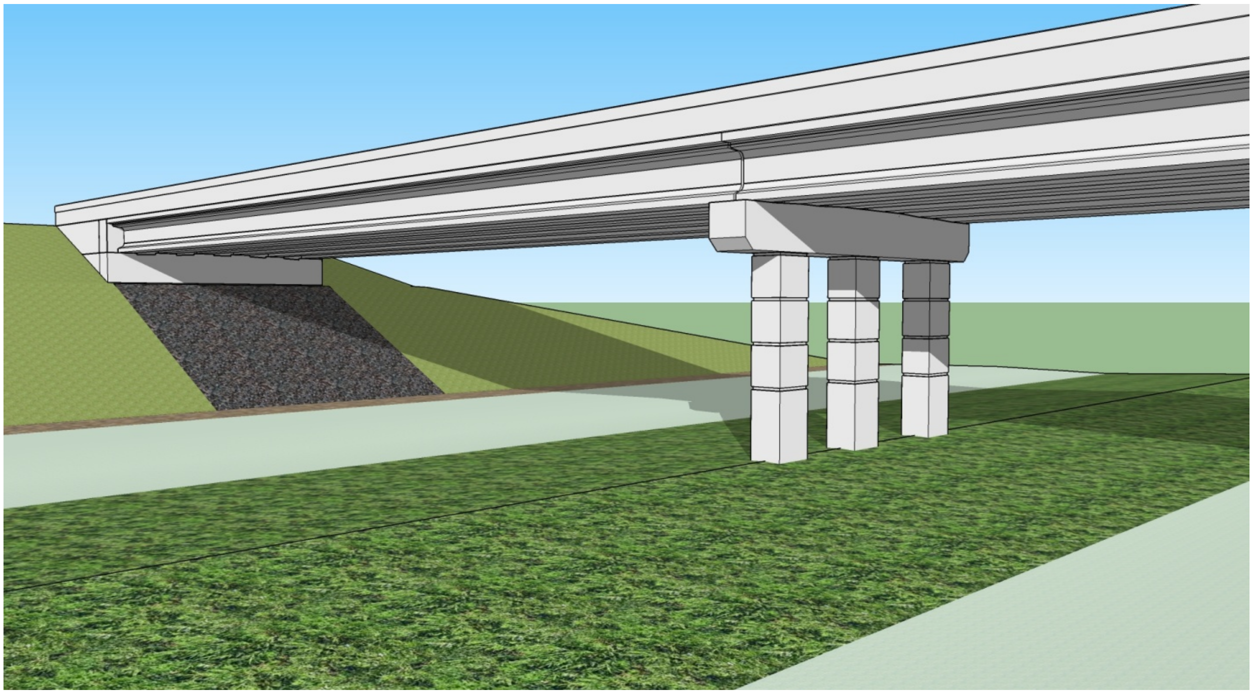

Figure 4.9-1
Aesthetic Concept Type I

- Plain abutment wings
- Single banded pier rustications
- Standard parapets
- Most rural and some urban applications

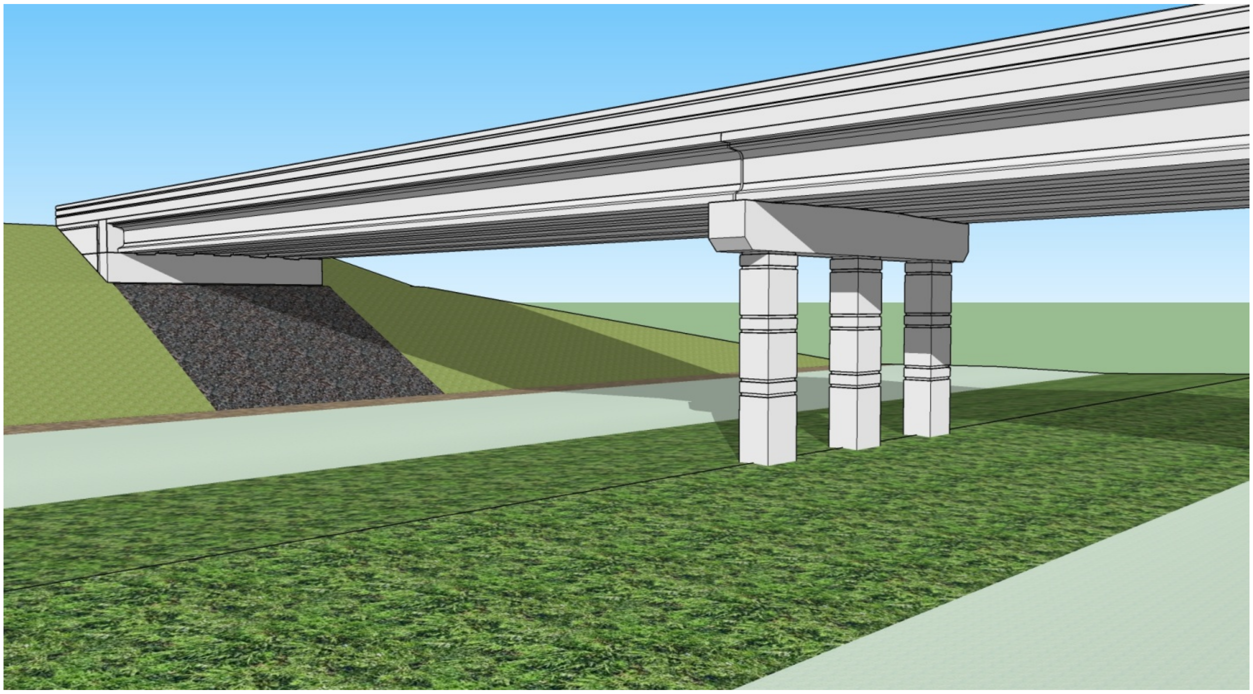

Figure 4.9-3
Aesthetic Concept Type II

- Rustication trim line on abutment wing
- Single or double banded pier rustications
- Rustication trim line(s) on parapets (one on 32” parapet and two on 42” parapet)
- Urban and other select applications

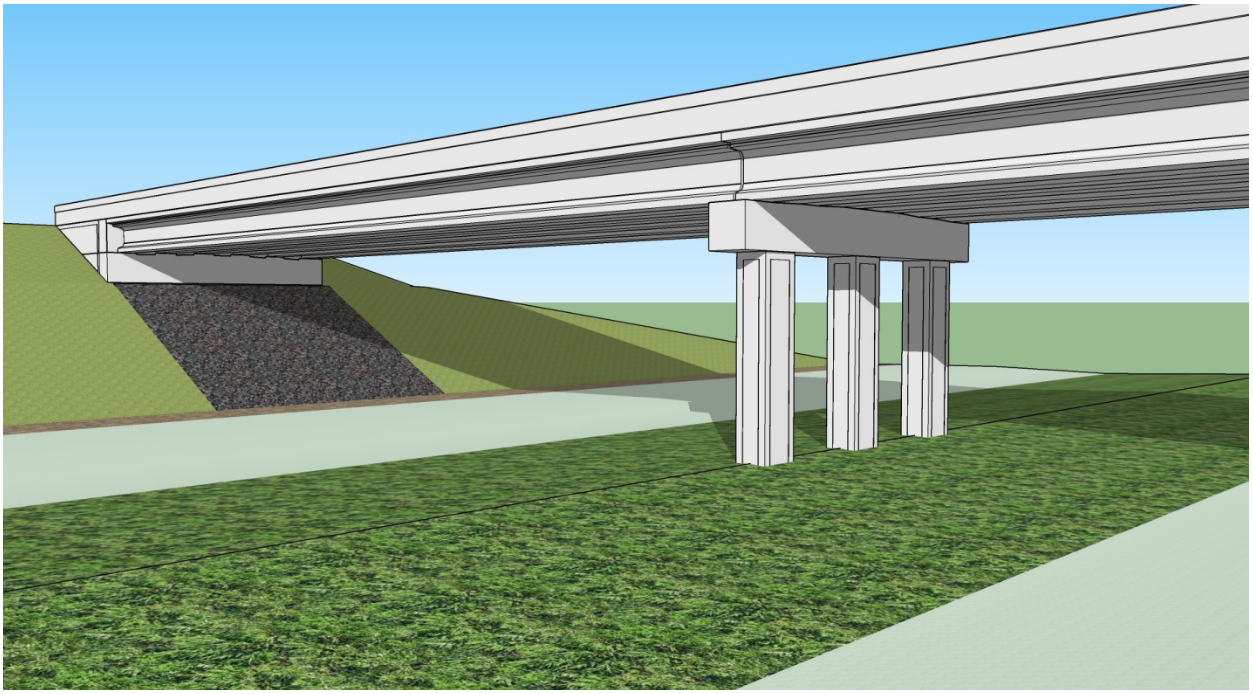

Figure 4.9-2
Aesthetic Concept Type III

- Recessed panel abutment wings
- Recessed panel columns
- Standard parapet
- Urban applications

4.10 References

1. AASHTO, *Bridge Aesthetics Sourcebook,* 2010.

2. Gottemoeller, Frederick, *Bridgescape: The Art of Designing Bridges*, John Wiley & Sons, Inc., 2004.

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5.1 Factors Governing Bridge Costs

Bridge costs are tabulated based on the bids received for all bridges let to contract. While these costs indicate some trends, they do not reflect all the factors that affect the final bridge cost. Each bridge has its own conditions which affect the cost at the time a contract is let. Some factors governing bridge costs are:

1. Location - rural or urban, or remote regions
2. Type of crossing
3. Type of superstructure
4. Skew of bridge
5. Bridge on horizontal curve
6. Type of foundation
7. Type and height of piers
8. Depth and velocity of water
9. Type of abutment
10. Ease of falsework erection
11. Need for special equipment
12. Need for maintaining traffic during construction
13. Limit on construction time
14. Complex forming costs and design details
15. Span arrangements, beam spacing, etc.

Figure 5.2-1 shows the economic span lengths of various type structures based on average conditions. Refer to Chapter 17 for discussion on selecting the type of superstructure.

Annual unit bridge costs are included in this chapter. The area of bridge is from back to back of abutments and out to out of the concrete superstructure. Costs are based only on the bridges let to contract during the period. In using these cost reports exercise care when a small number of bridges are reported as these costs may not be representative.

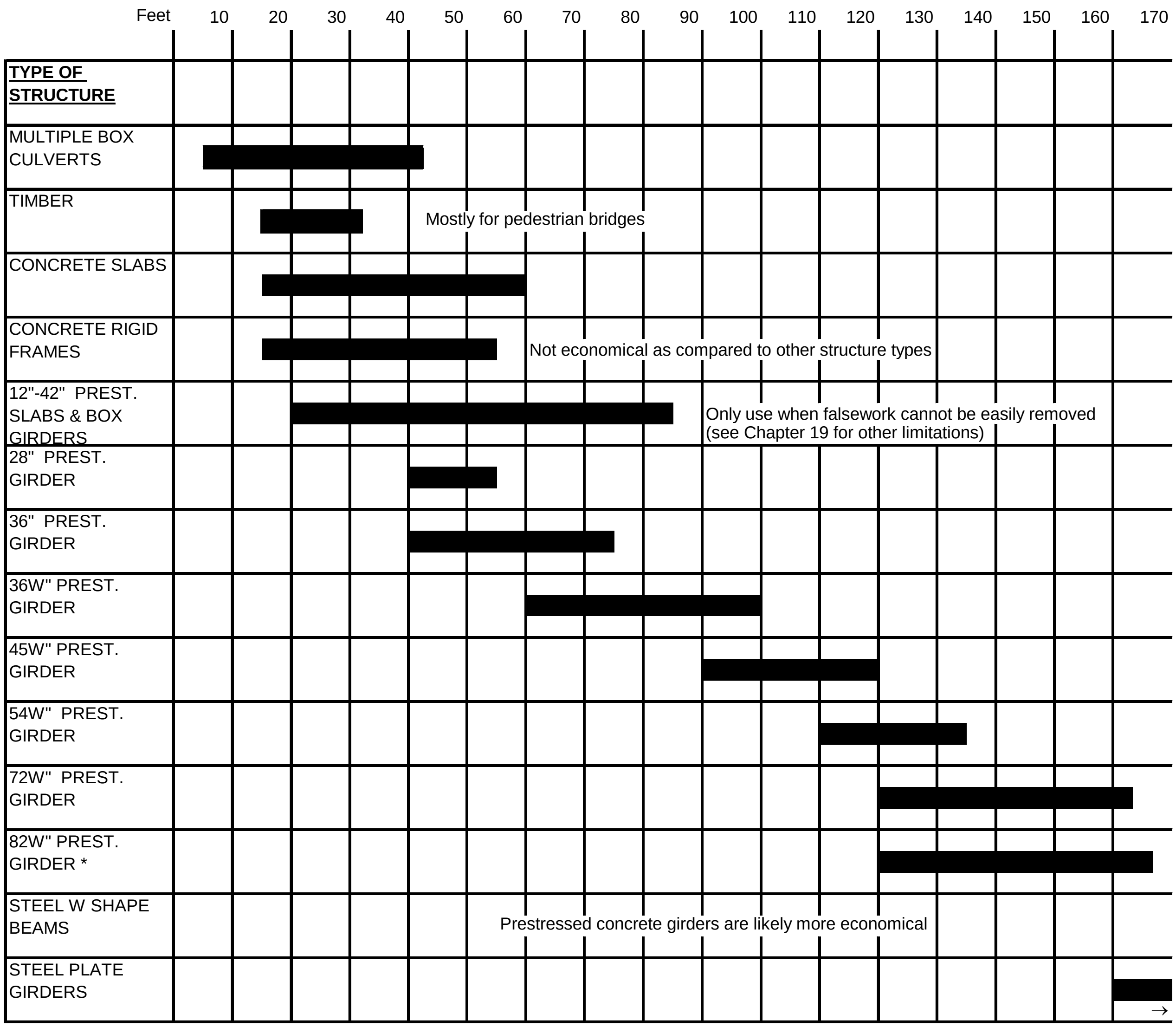
In these reports prestressed girder costs are grouped together because there is a small cost difference between girder sizes. Refer to unit costs. Concrete slab costs are also grouped together for this reason.

No costs are shown for rolled steel sections as these structures are not built very often. They have been replaced with prestressed girders which are usually more economical. The cost of plate girders is used to estimate rolled section costs.

For structures over a railroad, use the costs of grade separation structures. Costs vary considerably for railroad structures over a highway due to different railroad specifications.

Other available estimating tools such as *AASHTOWare Project Estimator* and *Bid Express*, as described in FDM 19-5-5, should be the primary tools for structure project cost estimations. Information in this chapter can be used as a supplemental tool.

5.2 Economic Span Lengths



*Currently there is a moratorium on the use of 82W" prestressed girders in Wisconsin

Figure 5.2-1
Economic Span Lengths

5.3 Contract Unit Bid Prices

Item No.	Bid Item	Unit	Cost
502.3100	Expansion Device (structure) (LS)	LF	210.97
502.3110.S	Expansion Device Modular (structure) (LS)	LF	969.95
SPV.0105	Expansion Device Modular LRFD (structure) (LS)	LF	1,947.75
513.2000	Railing Pipe (structure)	LF	137.02
513.4055	Railing Tubular Type H (structure) (LS)	LF	135.14
513.4060	Railing Tubular Type M (structure) (LS)	LF	228.70
513.4065	Railing Tubular Type PF (structure) (LS)	LF	179.96
513.4090	Railing Tubular Screening (structure) (LS)	LF	133.78
513.7005	Railing Steel Type C1 (structure) (LS)	LF	151.56
513.7010	Railing Steel Type C2 (structure) (LS)	LF	109.13
513.7015	Railing Steel Type C3 (structure) (LS)	LF	122.92
513.7020	Railing Steel Type C4 (structure) (LS)	LF	147.39
	Railing Steel Type C2 Pedestrian (structure) (LS)	LF	184.21
	Railing Steel Type C3 Pedestrian (structure) (LS)	LF	158.93
	Railing Steel Type C4 Pedestrian (structure) (LS)	LF	150.00
513.7050	Railing Type W (structure) (LS)	LF	120.30
513.7084	Railing Steel Type NY4 (structure)	LF	317.83

Table 5.3-1

Contract Unit Bid Prices for Structures - 2015

Other bid items should be looked up in Estimator or Bid Express

5.4Bid Letting Cost Data

This section includes past information on bid letting costs per structure type. Values are presented by structure type and include: number of structures, total area, total cost, superstructure cost per square foot and total cost per square foot.

5.4.1 2011 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	36	218,311	18,719,353	50.45	85.75
Reinf. Conc. Slabs (All but A5)	22	63,846	7,135,430	52.90	111.76
Reinf. Conc. Slabs (A5 Abuts)	14	21,005	2,470,129	53.00	117.60

Table 5.4-1
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	44	337,346	31,596,585	65.90	93.66
Reinf. Conc. Slabs (All but A5)	6	33,787	3,462,995	52.90	102.49

Table 5.4-2
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	5	2,140.00
Twin Cell	6	1,998.00
Triple Cell	5	3,518.00
Precast	1	7,385.00

Table 5.4-3
Box Culverts

Railroad Bridge	Cost per Sq. Ft.
B-20-210	3,654.30

Table 5.4-4
Railroad Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	6	7,893	494,274	62.62
MSE Panel Walls	19	87,000	6,679,782	76.78
Concrete Walls	3	3,516	237,230	67.47
Panel Walls	2	14,832	3,458,722	233.19
Tangent Pile Walls	3	10,139	1,581,071	155.94
Wire Faced MSE Walls	18	149,735	11,412,474	76.22
Soldier Pile Walls	2	7,849	779,563	99.32

Table 5.4-5
Retaining Walls

5.4.2 2012 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	18	115,512	11,610,435	53.88	100.50
Reinf. Conc. Slabs (All but A5)	22	80,797	8,269,942	53.04	102.35
Reinf. Conc. Slabs (A5 Abuts)	3	6,438	739,983	53.24	114.95

Table 5.4-6
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	58	697,381	65,044,526	65.91	93.27
Reinf. Conc. Slabs (All but A5)	1	5,812	491,683	43.73	84.60

Table 5.4-7
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	5	1,516.50
Twin Cell	6	3,292.00
Triple Cell	5	2,624.60
Precast	1	--

Table 5.4-8
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
B-40-761/762	325.22

Table 5.4-9
Pre-Fabricated Pedestrian Bridges

Pedestrian Bridge	Cost per Sq. Ft.
B-53-265	91.93

Table 5.4-10
Pedestrian Bridges

Buried Slab Bridge	Cost per Sq. Ft.
C-13-155	170.77

Table 5.4-11
Buried Slab Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	17	30,536	1,604,280	52.54
MSE Panel Walls	25	111,365	7,215,980	64.80
Modular Walls	1	500	49,275	98.50
Concrete Walls	2	5,061	416,963	82.39
Panel Walls	2	6,476	1,094,638	169.03
Wire Faced MSE Walls	21	109,278	16,130,424	147.61
Secant Pile Walls	1	12,545	2,073,665	165.30
Soldier Pile Walls	2	4,450	298,547	66.49
MSE Gravity Walls	1	975	61,470	63.05
Steel Sheet Pile Walls	5	8,272	352,938	42.67

Table 5.4-12
Retaining Walls

5.4.3 2013 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	17	120,700	12,295,720	49.75	101.87
Reinf. Conc. Slabs (All but A5)	12	26,361	2,244,395	48.26	85.14
Reinf. Conc. Slabs (A5 Abuts)	5	8,899	992,966	49.28	111.58

Table 5.4-13
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	52	672,482	67,865,859	69.67	100.92
Steel Plate Girders	6	195,462	27,809,905	89.62	142.28
Trapezoidal Steel Box Girders	7	571,326	98,535,301	116.21	172.47

Table 5.4-14
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	11	1,853.00
Twin Cell	5	2,225.00
Precast	3	1,079.00

Table 5.4-15
Box Culverts

Pre-Fab Pedestrian Bridge	Cost per Sq. Ft.
B-13-666	240.30
B-17-211	174.33

Table 5.4-16
Pre-Fabricated Pedestrian Bridges

Pedestrian Bridge	Cost per Sq. Ft.
B-13-661	222.06
B-13-656	105.60
B-13-657	106.62
B-40-784	289.02

Table 5.4-17
Pedestrian Bridges

Buried Slab Bridge	Cost per Sq. Ft.
B-24-40	182.28
B-5-403	165.57
B-13-654	210.68

Table 5.4-18
Buried Slab Bridges

Railroad Bridge	Cost per Sq. Ft.
B-40-773	1,151.00
B-40-774	1,541.00

Table 5.4-19
Railroad Bridges

Inverted T Bridge	Cost per Sq. Ft.
B-13-608	192.75
B-13-609	235.01
B-40-89	528.81

Table 5.4-20
Inverted T Bridges

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	8	13,351	447,017	33.48
MSE Panel Walls	55	255,817	23,968,072	93.69
Concrete Walls	23	32,714	2,991,867	91.46
Panel Walls	7	39,495	8,028,652	203.28
Wired Faced MSE Walls	28	160,296	20,554,507	128.17

Table 5.4-21
Retaining Walls

5.4.4 2014 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	20	457,537	52,424,589	53.80	114.58
Reinf. Conc. Slabs (All but A5)	27	59,522	8,104,551	58.89	136.16
Reinf. Conc. Slabs (A5 Abuts)	9	16,909	2,150,609	56.13	127.19
Buried Slab Bridges	1	4,020	198,583	11.63	49.40

Table 5.4-22
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	29	409,929	44,335,036	64.66	108.15
Reinf. Conc. Slabs (All but A5)	2	15,072	1,739,440	47.68	115.41
Steel Plate Girders	3	85,715	15,669,789	114.08	182.81
Steel I-Beam	1	2,078	596,712	82.99	287.16
Pedestrian Bridges	3	35,591	7,436,429	--	208.94
Trapezoidal Steel Box Girders	1	59,128	9,007,289	121.00	152.34

Table 5.4-23
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	10	2,361.30
Twin Cell	4	2,584.21
Triple Cell	1	2,928.40
Triple Pipe	1	1,539.41

Table 5.4-24
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	11	13,856	755,911	54.55
MSE Panel Walls	36	319,463	23,964,444	75.01
Concrete Walls	7	58,238	8,604,747	147.75
Panel Walls	1	3,640	590,682	162.28
Wired Faced MSE Walls	2	3,747	537,173	143.36
Secant Pile Walls	1	68,326	7,488,658	109.60
Soldier Pile Walls	9	33,927	4,470,908	131.78
Steel Sheet Pile Walls	2	3,495	159,798	45.72

Table 5.4-25
Retaining Walls

Noise Walls	Total Area (Sq. Ft)	Total Costs	Cost per Sq. Ft.
13	200,750	5,542,533	27.61

Table 5.4-26
Noise Walls

5.4.5 2015 Year End Structure Costs

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	22	338,229	41,220,154	60.96	121.87
Reinf. Conc. Slabs (Flat)	26	47,766	7,151,136	62.77	149.71
Reinf. Conc. Slabs (Haunched)	6	27,967	3,517,913	57.49	125.79
Buried Slab Bridges	1	2,610	401,000	43.74	153.64
Pre-Fab Pedestrian Bridges	3	29,304	3,440,091	--	117.39

Table 5.4-27
Stream Crossing Structures

Structure Type	No. of Bridges	Total Area (Sq. Ft.)	Total Costs	Super. Only Cost Per Square Foot	Cost per Square Foot
Prestressed Concrete Girders	58	768,458	102,067,913	66.04	132.82
Reinf. Conc. Slabs (Flat)	2	8,566	922,866	46.36	107.74
Reinf. Conc. Slabs (Haunched)	1	6,484	868,845	41.26	133.99
Steel Plate Girders	4	100,589	20,248,653	137.13	201.30
Trapezoidal Steel Box Girders	4	305,812	79,580,033	189.24	260.23
Rigid Frames	2	7,657	2,730,308	--	356.58
Timber	1	16,800	1,982,669	--	118.02
Pre-Fab Pedestrian Bridges	1	1,851	449,475	--	242.83

Table 5.4-28
Grade Separation Structures

Box Culvert Type	No. of Culverts	Cost per Lin. Ft.
Single Cell	2	2,235.67
Twin Cell	6	3,913.05
Single Pipe	1	2,262.11
Twin Pipe	2	426.20
Triple Pipe	2	1,424.09
Quadruple Pipe	1	2,332.96

Table 5.4-29
Box Culverts

Retaining Wall Type	No. of Walls	Total Area (Sq. Ft.)	Total Costs	Cost per Square Foot
MSE Block Walls	11	22,353	1,594,171	71.32
MSE Panel Walls	51	315,440	28,038,238	88.89
Wire Faced MSE Walls	3	10,345	1,501,948	145.19
Wired Faced MSE Walls w/ Precast Conc. Wall Panels	12	50,670	10,195,161	201.21
Secant Pile Walls	1	5,796.50	1,075,785	185.59
Soldier Pile Walls	6	37,498	6,037,788	161.02
Steel Sheet Pile Walls	6	11,319	668,227	59.04
Concrete Walls	2	6,850	712,085	103.96
MSE Panel Walls w/Integral Barrier	4	14,330	1,098,649	76.67

Table 5.4-30
Retaining Walls

Sign Structure Type		No. of Structures	Total Lineal Ft. of Arm	Total Costs	Cost per Lin. Ft.
Butterfly (1-Sign)	Conc. Col.	2	44	122,565	2,785.56
	1-Steel Col.	2	42	63,965	1,522.98
Butterfly (2-Signs)	1-Steel Col.	1	21	48,971	2,331.97
Cantilever	Conc. Col	18	530	1,217,454	2,297.08
	1-Steel Col.	15	394	528,950	1,342.85
Full Span	Conc. Col.	44	4,035	5,309,906	1,315.96
	1-Steel Col.	12	720	476,598	662.00
	2-Steel Col.	10	711	775,858	1,091.22
Full Span + Cantilever	Conc. Col.	1	84	166,003	1,976.22

Table 5.4-31
Sign Structures

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6.1 Approvals, Distribution and Work Flow

Production of Structural Plans

Regional Office	Prepare Structure Survey Report.
Geotechnical Section (Bur. of Tech. Services)	Make site investigation and prepare Site Investigation Report. See 6.2.1 for exceptions.
Structures Development Sect. (BOS)	Record Structure Survey Report.
Structures Design Section (BOS)	Determine type of structure.
	Perform hydraulic analysis if required.
	Check roadway geometrics and vertical clearance.
	Review Site Investigation Report and determine foundation requirements. Develop scour computations for bridges and record scour code on the preliminary plans.
	Draft preliminary plan layout of structure.
	Send copies of preliminary plans to Regional Office.
	If Federal aid funding is involved, send copies of preliminary plans to the Federal Highway Administration for major, moveable, and unusual bridges.
	If a waterbody that qualifies as a “navigable water of the United States” is crossed, a Permit drawing to construct the bridge is sent to the Coast Guard. If FHWA determines that a Coast Guard permit is needed, send a Permit drawing to the Coast Guard. If Federal aid is involved, preliminary plans are sent to

	the Federal Highway Administration for approval.
	Review Regional Office comments and other agency comments, modify preliminary plans as necessary.
	Review and record project for final structural plan preparation.
Structures Design Units (BOS)	Prior to starting project, Designer contacts Regional Office to verify preliminary structure geometry, alignment, width and the presence of utilities.
	Prepare and complete plans, specs and estimates for the specified structure.
	Give completed job to the Supervisor of Structures Design Unit.
Supervisor, Structures Design Unit (BOS)	Review plans, specs and estimates.
	Send copies of final structural plans and special provisions to Regional Offices.
	Sign lead structural plan sheet.
	Deliver final structural plans and special provisions to the Bureau of Project Development.
Bur. of Project Development	Prepare final approved structural plans for pre-contract administration.

See FEM Section 21-30-1.3 for information on determining whether a bridge crossing falls under the Coast Guard's jurisdiction.

6.2 Preliminary Plans

6.2.1 Structure Survey Report

The Structure Survey Report is prepared by Regional Office or consultant personnel to request a structure improvement project. The following forms in word format are used and are available at: http://www.dot.wisconsin.gov/forms/index.htm

Under the "Plans and Projects" heading:

DT1694	Separation Structure Survey Report
DT1696	Rehabilitation Structure Survey Report
DT1698	Stream Crossing Structure Survey Report (use for Culverts also)

The front of the form lists the supplemental information to be included with the report. Duplicate reports and supplemental information are required for Federal aid primary and Interstate projects.

6.2.1.1 BOS-Designed Structures

When preparing the Structure Survey Report, the region or consultant roadway designers will make their best estimate of structure type and location of substructure units. The completed Structure Survey Report with the locations of the substructure units and all required attachments and supporting information will then be submitted to the Bureau of Structures via e-submit (as "BOS Design") and also to the Geotechnical Section, through the Regional Soils Engineer. This submittal will take place a minimum of 18 months in advance of the earliest PS&E due date shown on the Structure Survey Report. The Geotechnical Section is responsible for scheduling and conducting the necessary soil borings. The Bureau of Structures and the Geotechnical Section will coordinate activities to deliver the completed structure plans on schedule.

When a geotechnical consultant is performing the subsurface exploration, the work typically proceeds after the preliminary plans have been assembled by the Bureau of Structures. Under some circumstances, it may be expected that the geotechnical information gathered will be included in the Structure Survey Report in advance of the development of the preliminary plans. In the case of the Geotechnical Section performing the subsurface exploration, the geotechnical work will proceed after the preliminary plans have been assembled by the Bureau of Structures.

The Project Manager may request information on structure type and substructure locations from the Bureau of Structures if such information is necessary to expedite the environmental process.

Under this process, the scheduling of geotechnical work is coordinated with the Bureau of Structures toward completion of the bridge plans by the final plan due date. If other geotechnical work is required for the project, the Project Manager should coordinate with the Regional Soils Engineer and the Geotechnical Section to promote efficiency of field drilling operations.

If the preliminary plans are required more than one year in advance of the final plan due date due to the unique needs of the project, the Project Manager should discuss this situation with the Bureau of Structures Design Supervisor prior to submitting the Structure Survey Report.

Coordination early in the design process with DNR regarding removal techniques for the existing structure (if applicable), and new structure placement and type is very important. The status of any agreements with the DNR, which affect the structure, should be noted under additional information on the Structure Survey Report.

6.2.1.2 Consultant-Designed Structures

When preparing the Structure Survey Report, the region or consultant roadway designer's responsibility for submitting the Structure Survey Report depends on their involvement with the design of the structure and the soils investigation. Refer to Table 30.1 in FDM 3-20-30.2.2 for the process involved with differing levels of involvement.

If the preliminary bridge plans are required more than one year in advance of the final plan due to the unique needs of the project, the Project Manager should discuss this situation with the consultant.

Coordination early in the design process with DNR regarding removal techniques for the existing structure (if applicable), and new structure placement and type is very important. The status of any agreements with the DNR, which affect the structure, should be noted under additional information on the Structure Survey Report.

6.2.2 Preliminary Layout

6.2.2.1 General

The preparation of a preliminary layout for structures is primarily for the purpose of presenting an exhibit to the agencies involved for approval, before proceeding with final design and preparation of detail plans. When all the required approvals are obtained, the preliminary layout is used as a guide for final design and plan preparation.

The drawings for preliminary layouts are on sheets having an overall width of 11 inches and an overall length of 17 inches and should be placed within the current sheet border under the #8 tab.

6.2.2.2 Basic Considerations

The following criteria are used for the preparation of preliminary plans.

1. Selection of Structure Type. Refer to Chapter 17 - Superstructure-General, for a discussion of structure types.

2. Span Arrangements. For stream crossings the desired minimum vertical clearance from high water to low chord is given in Chapter 8 - Hydraulics. Span lengths for multiple span stream crossings are in most cases a matter of economics and the provision for an opening that adequately passes flood flows, ice and debris. For structures over waterways that qualify as navigable waters of the United States, the minimum vertical and horizontal clearances of the navigable span are determined by the U.S. Coast Guard after considering the interests of both highway and waterway transportation users.

   For most of the ordinary grade separation structures the requirements for horizontal clearance determine the span arrangements. Refer to Chapter 17 - Superstructure-General for span length criteria.

3. Economics.

   Economy is a primary consideration in determining the type of structure to be used. Refer to Chapter 5 – Economics and Costs, for cost data.

   At some stream crossings where the grade line permits considerable head room, investigate the economy of a concrete box culvert versus a bridge type structure. When economy is not a factor, the box culvert is the preferred type from the standpoint of maintenance costs, highway safety, flexibility for roadway construction, and provision of a facility without roadway width restrictions.

4. Aesthetics. Recognition of aesthetics as an integral part of a structure is essential if bridges are to be designed in harmony with adjacent land use and development. Refer to Chapter 4 - Aesthetics.

5. Hydraulic Consideration. Stream crossing structures are influenced by stream flow, drift, scour, channel conditions, ice, navigation, and conservation requirements. This information is submitted as part of the Structure Survey Report. Refer to Chapter 8 - Hydraulics for Hydraulic considerations and Section 8.1.5 for Temporary Structure Criteria.

6. Geometrics of Design. The vertical and horizontal clearance roadway widths, design live loading, alignment, and other pertinent geometric requirements are given in Chapter 3.

7. Maintenance. All bridge types require structural maintenance during their service life. Maintenance of approaches, embankments, drainage, substructure, concrete deck, and minor facilities is the same for the various types of bridges. A minimum draining grade of 0.5% across the bridge is desirable to eliminate small ponds on the deck except for open railings where the cross slope is adequate.

   Epoxy coated bar steel is required in all new decks and slabs.

Steel girders require periodic painting unless a type of weathering steel is used. Even this steel may require painting near the joints. It is more difficult to repaint steel girders that span busy highways.

Cast-in-place reinforced concrete box girders and voided slabs have a poor experience in Wisconsin. They should not be used on new structures.

Deck expansion joints have proved to be a source of maintenance problems. Bridges designed with a limited number of watertight expansion devices are recommended.

8. Construction. Occasionally a structure is proposed over an existing highway on which traffic must be maintained. If the roadway underneath carries high volumes of traffic, any obstruction such as falsework would be hazardous as well as placing undesirable vertical clearance restrictions on the traveled way. This is also true for structures over a railroad.

   For structures over most high-volume roadways construction time, future maintenance requirements, and provision for future expansion of the roadway width, have considerable influence on the selection of the final product.

9. Foundations. Poor foundation conditions may influence the structure geometry. It may be more economical to use longer spans and fewer substructure units or a longer structure to avoid high approach fills.

10. Environmental Considerations. In addition to the criteria listed above all highway structures must blend with the existing site conditions in a manner that is not detrimental to environmental factors. Preservation of fish and wildlife, pollution of waters, and the effects on surrounding property are of primary concern in protecting the environment. The design of structures and the treatment of embankments must consider these factors.

11. Safety. Safety is a prime consideration for all aspects of the structure design and layout. Bridge railings are approved through actual vehicle crash testing.

6.2.2.3 Requirements of Drawing

6.2.2.3.1 Plan View

The plan view is preferably placed in the upper left-hand portion of the sheet at the largest scale practical (1"=10') and shows the following basic information:

1. The plan view shall be shown with the reference line stationing progressing upstation from left to right on the sheet. A reference north arrow shall be included.

2. Structure span lengths, (center-to-center of piers and to centerline of bearing at abutments, end distance from centerline of bearing to back face of abutment and overall length of structure).

3. Dimensions along the reference line except for structures on a curve in which case they are along a tangent to the curve.

4. Stations are required at centerline of piers, centerline of bearing at abutments, and end of deck or slab.

5. Stations at intersection with reference line of roadway underneath for grade separation structures.

6. Direction of stationing increase for highway or railroad beneath a structure.

7. Detail the extent of slope paving or riprap.

8. Direction of stream flow and name if a stream crossing.

9. Highway number and direction and number of traffic lanes.

10. Horizontal clearance dimensions, pavement, shoulder, sidewalk, and structure roadway widths.

11. Median width if dual highway.

12. Skew angles and angles of intersection with other highways, streets or railroads.

13. Horizontal curve data if within the limits of the structure showing station of P.C., P.T., and P.I. Complete curve data of all horizontal curves which may influence layout of structure.

14. Location and dimension of minimum vertical clearance for highway or railroad grade separation structures.

    a. The minimum vertical clearance should be noted as the “Point of Minimum Vertical Clearance” for all spans.

    b. Minimum vertical clearance is required for each span of a structure above a traveled way (i.e., roadway, railroad track, etc.).

    c. Refer to *Facilities Development Manual 11-35-1, Section 1.5* for guidance pertaining to the required locations to be checked for underclearance.

15. If floor drains are proposed the type, approximate spacing, and whether downspouts are to be used.

16. Existing structure description, number, station at each end, buildings, underground utilities and pole lines giving owner's name and whether to remain in place, be relocated or abandoned.

17. Indicate which wingwalls have beam guard rail attached if any and wing lengths.

18. Structure numbers on plan.

19. Excavation protection for railroads.

20. Limits of railroad right-of-way. The locations are for reference only and need not be dimensioned.

21. Location of deck lighting or utilities if any.

22. Name Plate location.

23. Bench Mark Cap Location

24. Locations of surface drains on approach pavement.

25. Tangent offsets between reference line and tangent line along C_L substructure unit. Also include tangent offsets for edge of deck and reference line at 10 foot intervals.

6.2.2.3.2 Elevation View

The elevation view is preferably placed below the plan view. If the structure is not skewed the substructure units are to be a straight projection from the plan view. If skewed, the elevation is a view normal to substructure units. The view shows the following basic information:

1. Profile of existing groundline or streambed.

2. Cross-section of highway or channel below showing back slopes at abutments.

3. Elevation of top of berm and rate of back slope used in figuring length of structure.

4. Type and extent of slope paving or riprap on back slopes.

5. Proposed elevations of bottom of footings and type of piling if required.

6. Depth of footings for piers of stream crossing and if a seal is required, show and indicate by a note.

7. Location and dimension of minimum vertical clearance.

    a. Minimum vertical clearance is required for each span of a structure above a traveled way (i.e., roadway, railroad track, etc.).

    b. Refer to *Facilities Development Manual 11-35-1, Section 1.5* for guidance pertaining to the required locations to be checked for underclearance.

8. Streambed, observed and high water elevations for stream crossings.

9. Location of underground utilities, with size, kind of material and elevation indicated.

10. Location of fixed and expansion bearings.

11. Location and type of expansion devices.

12. Limits of railroad right-of-way. The locations are for reference only and need not be dimensioned.

An elevation view is required for deck replacements, overlays with full-depth deck repair and painting plans (or any rehabilitation requiring the contractor to go beneath the bridge). Enough detail should be given to provide the contractor an understanding of what is beneath the bridge (e.g. roadway, bike path, stream, type of slope paving, etc.).

6.2.2.3.3 Cross-Section View

A view of a typical pier is shown as a part of the cross-section. The view shows the following general information:

1. Slab thickness, curb height and width, type of railing.
2. Horizontal dimensions tied into a reference line or centerline of roadway.
3. Girder spacing with girder depth.
4. Direction and amount of crown or superelevation, given in %.
5. Point referred to on profile grade.
6. Type of pier with size and number of columns proposed.
7. For solid, hammerhead or other type pier approximate size to scale.
8. Dimension minimum depth of bottom of footings below ditch or finished ground line or if railroad crossing below top of rail.
9. Location for public and private utilities to be carried in the superstructure. Label owner's name of utilities.
10. Location of lighting on the deck or under the deck if any.

6.2.2.3.4 Other Requirements

1. Profile grade line across structure showing tangent grades and length of vertical curve. Station and elevation of P.C., P.I., P.T., and centerline of all substructure units.

   Profile grade line of highway beneath structure if highway separation or of top rail if railroad separation. Stations along top of rail are to be tied into actual stationing as established by the railroad company.
2. Channel change section if applicable. Approximate stream bed elevation at low point.
3. Any other view or detail which may influence the bridge type, length or clearance.
4. List design data including:

Material Properties:

- Concrete Superstructure
- Concrete Substructure
- Bar Steel Reinforcement
- Structural Steel
- Prestressed Concrete
- Prestressing Steel

*Note: For rehabilitation projects, include Material Properties only for those materials utilized in the rehabilitation.

Foundations

- Soil Bearing Pressure
- Pile Type and Capacity (see 6.3.2.1)

Ratings (Plans Including Ratings that have been changed)

Live Load:

Design Loading: HL-93

Inventory Rating Factor: RF = X.XX

Operating Rating Factor: RF = X.XX

Wisconsin Standard Permit Vehicle (Wis-SPV) = XXX kips

(See Chapter 45 – Bridge Rating (45.8.2) for additional information)

Ratings (Plans Including Ratings that have not been changed)

Live Load:

Design Loading: HL-93 (taken from HSI, xx/xx/2xxx)

Inventory Rating Factor: RF = X.XX (taken from HSI, xx/xx/2xxx)

Operating Rating Factor: RF = X.XX (taken from HSI, xx/xx/2xxx)

Wisconsin Standard Permit Vehicle (Wis-SPV) = XXX kips (taken from HSI, xx/xx/2xxx)

Hydraulic Data

Base Flood

- 100 Year Discharge
- Stream Velocity through proposed bridge
- 100 Year Highwater Elevation
- 2 Year Flow & 2 year Highwater Elevation (Based on new structure opening)
- Waterway Area
- Drainage Area
- Scour Code

Overtopping Flood   OR (Overtopping N/A, for Floods > the 100 Year Flood)

- Overtopping Frequency
- Overtopping Elevation
- Overtopping Discharge

5. Show traffic data. Give traffic count, data and highway for each highway on grade separation or interchange structure.

6.2.2.4 Utilities

In urban areas, public and private utilities generally have their facilities such as sewers, water cables, pipes, ducts, etc., underground, or at river crossings, in the streambed.

If these facilities cannot be relocated, they may interfere with the most economical span arrangement. The preferred location of light poles is at the abutments or piers.

Overhead power lines may cause construction problems or maintenance inspection problems. Verify if they exist and notify Utilities & Access Management Unit (Bureau of Tech. Services) to have them removed.

It is the general policy to not place utilities on the structure. The Utilities & Access Management Unit approves all utility applications and determines whether utilities are placed on the structures or can be accommodated some other way. Refer all requests to them. Also see Chapter 18 of the FDM and Chapter 4 of "*WisDOT Guide to Utility Coordination*".

6.2.3 Distribution of Exhibits

6.2.3.1 Federal Highway Administration (FHWA).

FHWA memorandums "Implementing Guidance-Project Oversight under Section 1305 of the Transportation Equity Act for the 21st Century (TEA-21) of 1998" dated August 20, 1998, and "Project Oversight Unusual Bridges and Structures" dated November 13, 1998, indicate that **FHWA Headquarters Bridge Division or the Division Office must review and approve preliminary plans for unusual bridges and structures on the following projects:**

1. Projects on the Interstate System

2. Projects on the National Highway System (NHS) but not on the Interstate System, unless it is determined by FHWA and WisDOT that the responsibilities can be assumed by WisDOT

3. Projects on non-NHS Federal-aid highways, and eligible projects on public roads which are not Federal-aid highways if WisDOT determines that it is not appropriate for WisDOT to assume the responsibilities

Technical assistance is also available upon request for projects/structures that are not otherwise subject to FHWA oversight.

Unusual bridges have the following characteristics:

- Difficult or unique foundation problems
- New or complex designs with unique operational or design features
- Exceptionally long spans
- Design procedures that depart from currently recognized acceptable practices

Examples of unusual bridges:

- Cable-stayed
- Suspension
- Arch
- Segmental concrete
- Movable
- Truss
- Bridge types that deviate from AASHTO bridge design standards or AASHTO guide specifications for highway bridges

- Major bridges using load and resistance factor design specifications
- Bridges using a three-dimensional computer analysis
- Bridges with spans exceeding 350 feet

Examples of unusual structures:

- Tunnels
- Geotechnical structures featuring new or complex wall systems or ground improvement systems
- Hydraulic structures that involve complex stream stability countermeasures, or designs or design techniques that are atypical or unique

Timing of submittals is an important consideration for FHWA approval and assistance, therefore, **FHWA should be involved as early as possible.**

The following preliminary documents should be submitted electronically (PDF format) to FHWA:

1. Preliminary plans (Type, Size and Location)
2. Bridge/structures related environmental concerns and suggested mitigation measures
3. Studies of bridge types and span arrangements
4. Approach bridge span layout plans and profile sheets
5. Controlling vertical and horizontal clearance requirements
6. Roadway geometry
7. Design specifications used
8. Special design criteria
9. Cost estimates
10. Hydraulic and scour design studies/reports showing scour predictions and related mitigation measures
11. Geotechnical studies/reports
12. Information on substructure and foundation types

Note: Much of this information may be covered by the submittal of a Structure Type Selection Report.

6.2.3.2 Coast Guard

Permit application guides are published by the Coast Guard Districts. Consult FDM Section 21-30-1.3 for additional guidance.

6.2.3.3 Regions

A copy of the preliminary plans is sent to the Regional Office involved, for their review. For structures financed partially or wholly by a county, city, village or township, their approval should be obtained by the Regional Office and approval notice forwarded to the Bureau of Structures.

6.2.3.4 Utilities

For all structures which involve a railroad, four prints of the preliminary drawing are submitted to the Utilities & Access Management Unit for submission to the railroad company for approval.

If private or public utilities wish to make application to attach their facilities (water, and sewer mains, ducts, cables, etc.) to the structure, they must apply to the Utilities & Access Management Unit for approval.

6.2.3.5 Other Agencies

A copy of preliminary plans (preliminary layout, plan & profile, and contour map) for stream crossing bridges are forwarded to the Department of Natural Resources for comment, in accordance with the cooperative agreement between the Department of Transportation and the Department of Natural Resources. (See Chapter 8 - Hydraulics).

6.3 Final Plans

This section describes the general requirements for the preparation of construction plans for bridges, culverts, retaining walls and other related highway structures. It provides a standard procedure, form, and arrangement of the plans for uniformity.

6.3.1 General Requirements

6.3.1.1 Drawing Size

Sheets are 11 inches wide from top to bottom and 17 inches long. A border line is provided on the sheet 1 inch from the left edge, and ¼ inch from other edges. Title blocks are provided on the first sheet for a signature and other required information. The following sheets contain the same information without provision for a signature.

6.3.1.2 Scale

All drawings insofar as possible are drawn to scale. Such details as reinforcing steel, steel plate thicknesses, etc. are not scaled. The scale is adequate to show all necessary details.

6.3.1.3 Line Thickness

Object lines are the widest line on the drawing. Lines showing all or part of an existing structure or facility are shown by dashed lines of somewhat lighter weight.

Lines showing bar steel are lighter than object lines and are drawn continuous without any break. Dimension and extension lines are lighter than bar steel lines but heavy enough to make a good reproduction.

6.3.1.4 Lettering and Dimensions

All lettering is upper case. Lettering and dimensions are read from the bottom or right hand side and should be placed above the dimension lines. Notes and dimension text are 0.12 inches high; view titles are 0.20 inches high (based on full size sheet, 22" x 34"). Dimensions are given in feet and inches. Elevations are given in decimal form to the nearest 0.01 of a foot. Always show two decimal places. Although plan dimensions are very accurate, the contractor should use reasonable tolerances during construction of the project by building to the accuracy required. Detail structural steel to the thickness of the material involved.

6.3.1.5 Notes

Show any notes to make the required details clear on the plans. Do not include material that is part of the specifications.

6.3.1.6 Standard Insert Drawings

Standard detail sheets are available for railings and parapets, prestressed girders, bearings, expansion joints, and drains. Fill in the dimensions and titles required and insert in the final plans.

Standard insert sheets can be found at: http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/insert-sheets.aspx

6.3.1.7 Abbreviations

Abbreviations are to be used throughout the plans whenever possible. Abbreviations approved to be used are as follows:

Abutment	ABUT.	East	E.
Adjacent	ADJ.	Elevation	EL.
Alternate	ALT.	Estimated	EST.
And	&	Excavation	EXC.
Approximate	APPROX.	Expansion	EXP.
At	@	Fixed	F.
Back Face	B.F.	Flange Plate	Fl. Pl.
Base Line	B/L	Front Face	F.F.
Bench Mark	B.M.	Galvanized	GALV.
Bearing	BRG.	Gauge	GA.
Bituminous	BIT.	Girder	GIR.
Cast-in-Place	C.I.P.	Highway	HWY.
Centers	CTRS.	Horizontal	HORIZ.
Center Line	C/L	Inclusive	INCL.
Center to Center	C to C	Inlet	INL.
Column	COL.	Invert	INV.
Concrete	CONC.	Left	LT.
Construction	CONST.	Left Hand Forward	L.H.F.
Continuous	CONT.	Length of Curve	L.
Corrugated Metal Culvert Pipe	C.M.C.P.	Live Load	L.L.
Cross Section	X-SEC.	Longitudinal	LONGIT.
Dead Load	D.L.	Maximum	MAX.
Degree of Curve	D.	Minimum	MIN.
Degree	°	Miscellaneous	MISC.
Diaphragm	DIAPH.	North	N.
Diameter	DIA.	Number	NO.

Discharge	DISCH.	Near Side, Far Side	N.S.F.S.
Per Cent	%	Sidewalk	SDWK.
Plate	PL	South	S.
Point of Curvature	P.C.	Space	SPA.
Point of Intersection	P.I.	Specification	SPEC
Point of Tangency	P.T.	Standard	STD.
Point on Curvature	P.O.C.	Station	STA.
Point on Tangent	P.O.T.	Structural	STR.
Property Line	P.L.	Substructure	SUBST.
Quantity	QUAN.	Superstructure	SUPER.
Radius	R.	Surface	SURF.
Railroad	R.R.	Superelevation	S.E.
Railway	RY.	Symmetrical	SYM
Reference	REF.	Tangent Line	TAN. LN.
Reinforcement	REINF.	Transit Line	T/L
Reinforced Concrete Culvert Pipe	R.C.C.P.	Transverse	TRAN.
Required	REQ'D.	Variable	VAR.
Right	RT.	Vertical	VERT.
Right Hand Forward	R.H.F.	Vertical Curve	V.C.
Right of Way	R/W	Volume	VOL.
Roadway	RDWY.	West	W.
Round	ø	Zinc Gauge	ZN. GA.
Section	SEC.		

Table 6.3-1
Abbreviations

6.3.1.8 Nomenclature and Definitions

Universally accepted nomenclature and approved definitions are to be used wherever possible.

6.3.2 Plan Sheets

The following information describes the order of plan sheets and the material required on each sheet.

Plan sheets are placed in order of construction generally as follows:

1. General Plan
2. Subsurface Exploration
3. Abutments
4. Piers
5. Superstructure and Superstructure Details
6. Railing and Parapet Details

Show all views looking up station.

6.3.2.1 General Plan (Sheet 1)

See the BOS web page, CADD Resource Files, for the latest sheet borders to be used. Sheet borders are given for new bridges, rehabilitation projects and concrete box culverts. A superstructure replacement utilizing the existing substructure, bridge widenings, as well as damaged girder replacements should use the sheet border for a new structure. See Chapter 40 - Bridge Rehabilitation for criteria as to when superstructure replacements are allowed.

1. Plan View

   Same requirements as specified for preliminary drawing, except do not show contours of groundline and as noted below.

   a. Sufficient dimensions to layout structure in the field.

   b. Describe the structure with a simple note such as: Four span continuous steel girder structure.

   c. Station at end of deck on each end of bridge.

   On Structure Replacements

   Show existing structure in dashed-lines on Plan View.

2. Elevation View

   Same requirements as specified for preliminary plan except:

   a. Show elevation at bottom of all substructure units.

   b. Give estimated pile lengths where used.

3. Cross-Section View

   Same requirements as specified for preliminary plan except:

a. For railroad bridges show a railroad cross-section.

b. View of pier if the bridge has a pier (s), if not, view of abutment.

4. Grade Line

Same requirements as specified for preliminary plan.

5. Design and Traffic Data

Same requirements as specified for preliminary plans, plus see 6.3.2.1 for guidance regarding sheet border selection.

6. Hydraulic Information, if Applicable

7. Foundations

Give soil/rock bearing capacity or pile capacity.

Example for General Plan sheet: Abutments to be supported on HP 10 x 42 steel piling driven to a required driving resistance of 180 tons * per pile as determined by the Modified Gates Dynamic Formula. Estimated 50'-0" long.

*The factored axial resistance of piles in compression used for design is the required driving resistance multiplied by a resistance factor of 0.5 using modified Gates to determine driven pile capacity.

Abutments with spread footings to be supported on sound rock with a required factored bearing resistance of "XXX" PSF ***. A geotechnical engineer, with three days notice, will determine the factored bearing resistance by visual inspection prior to construction of the abutment footing.

*** The factored bearing resistance is the value used for design.

Repeat the note above on each substructure sheet, except the asterisk (*) and subsequent explanation of factored design resistance need not appear on individual substructure sheets.

See Table 11.3-5 for typical maximum driving resistance values.

8. Estimated Quantities

a. Enter bid items and quantities as they appear, and in the order in which they appear in the "Schedule of Bid Items" of the Standard Specifications. Put items not provided for at the bottom of the list. Enter quantities for each part of the structure, (superstructure, each abutment, each pier) under a separate column with a grand total.

Quantities are to be bid under items for the Structure Type and not by the "B" or "C" numbers. For example, concrete for a multi-cell box culvert exceeding a total length of 20 feet is to be bid under item Concrete Masonry Culverts. As another example, a bridge having a length less than 20 feet would be given a "C" number; however, the concrete bid item is Concrete Masonry Bridges.

b. For incidental items to be furnished for which there is no bid item, and compensation is not covered by the *Standard Specifications* or *Special Provisions*, note on the plans the most closely related bid item that is to include the cost in the price bid per unit of item. As an example, the cost of concrete inserts is to be included in the price bid per cubic yard of concrete masonry.

9. General Notes

A standard list of notes is given in 6.3.2.1.1 and 6.3.2.1.2. Use the notes that apply to the structure drawn on the plans.

10. List of Drawings

Each sheet is numbered sequentially beginning with 1 for the first sheet. Give the sheet number and title of sheet.

11. Bench Marks

Give the location, description and elevation of the nearest bench mark.

12. Title Block

Fill in all data for the Title Block except the signature. The title of this sheet is "General Plan". Use the line below the structure number to describe the type of crossing. (Example: STH 15 SB OVER FOX RIVER). See 6.3.2.1 for guidance regarding sheet border selection.

13. Professional Seal

All final bridge plans prepared by Consultants or Governmental Agencies shall be professionally sealed, signed, and dated on the general plan sheet.

This is not required for WisDOT prepared plans, as they are covered elsewhere.

6.3.2.1.1 Plan Notes for New Bridge Construction

1. Drawings shall not be scaled. Bar Steel Reinforcement shall be embedded 2” clear unless otherwise shown or noted.

2. All field connections shall be made with 3/4” diameter friction type high-tensile strength bolts unless shown or noted otherwise.

3. Slab falsework shall be supported on piles or the substructure unless an alternate method is approved by the Engineer.

4. The first or first two digits of the bar mark signifies the bar size.

5. The slope of the fill in front of the abutments shall be covered with heavy riprap and geotextile fabric Type ‘HR’ to the extent shown on sheet 1 and in the abutment details.

6. The slope of the fill in front of the abutments shall be covered with slope paving material to the extent shown on sheet 1 and in the abutment details.

7. The stream bed in front of the abutment shall be covered with riprap as shown on this sheet and in the abutment details.

8. The existing stream bed shall be used as the upper limits of excavation at the piers.

9. The existing ground line shall be used as the upper limits of excavation at the piers.

10. Within the length of the box all spaces excavated and not occupied by the new structure shall be backfilled with Structure Backfill to the elevation and section existing prior to excavation within the length of the culvert.

11. At the backface of abutment all volume which cannot be placed before abutment construction and is not occupied by the new structure shall be backfilled with structure backfill.

12. Concrete inserts to be furnished by the utility company and placed by the contractor. Cost of placing inserts shall be included in the bid price for concrete masonry.

13. Prestressed Girder Bridges - The haunch concrete quantity is based on the average haunch shown on the Prestressed Girder Details sheet.

14. The quantity for Backfill Structure, bid item 210.0100, is calculated based on the applicable Figures 12.6-1 and 12.6-2 in the Wisconsin Department of Transportation Bridge Manual.

6.3.2.1.2 Plan Notes for Bridge Rehabilitation

WisDOT policy item:

The note “Dimensions shown are based on the original structure plans” is acceptable. However, any note stating that the contractor shall field verify dimensions is not allowed.

It is the responsibility of the design engineer to use original structure plans, as-built structure plans, shop drawings, field surveys and structure inspection reports as appropriate when producing rehabilitation structure plans of any type (bridges, retaining walls, box culverts, sign structures, etc.). If uncertainty persists after reviewing available documentation, a field visit may be necessary by the design engineer.

1. Dimensions shown are based on the original structure plans.

2. All concrete removal not covered with a concrete overlay shall be defined by a 1 inch deep saw cut.

3. Utilize existing bar steel reinforcement where shown and extend 24 bar diameters into new work, unless specified otherwise.

4. Concrete expansion bolts and inserts to be furnished and placed by the contractor under the bid price for concrete masonry.

5. At "Curb Repair" expose existing reinforcement a minimum of 1 1/2" clear.

6. Existing floor drains to remain in place. Remove top of deck in drain area as directed by the Field Engineer to allow placing and sloping of 1 1/2" concrete overlay.

7. Expansion joint assembly, including anchor studs and hardware shall be paid for in the lump sum price bid as "Expansion Device B-_____" or "Expansion Device Modular B- ______".

8. Clean and fill existing longitudinal and transverse cracks with penetrating epoxy as directed by the Field Engineer.

9. Variations to the new grade line over 1/4" must be submitted by the Field Engineer to the Structures Design Section for review.

10. The contractor shall supply a new name plate in accordance with Section 502.3.11 of the *Standard Specifications* and the standard detail drawings. Name plate to show original construction year.

6.3.2.2 Subsurface Exploration

This sheet is initiated by the Geotechnical Engineer. The following information is required on the sheet. Bridge details are not drawn by the Geotechnical Engineer.

1. Plan View

   Show a plan layout of structure with survey lines, reference lines, pier and abutment locations and location of borings and probings plotted to scale.

   On box culvert structure plans, show three profile lines of the existing ground elevations (along the centerline and outer walls of the box). Scale the information for these lines from the site contour map that is a part of the structure survey report.

2. Elevation

   Show a centerline profile of existing ground elevation.

   Show only substructure units at proper elevation w/no elevations shown. Also show the pile lengths.

Show the kind of material, its located depth, and the blow count of the split spoon sampler for each boring. Give the blow count at about 5 foot intervals or where there is a significant change in material.

6.3.2.3 Abutments

Use as many sheets as necessary to show details clearly. Each substructure unit should have its own plan sheet(s). Show all bar steel required using standard notations; solid lines lengthwise and solid dots in cross section. Give dimensions for a skewed abutment to a reference line which passes through the intersection for the longitudinal structural reference line and centerline of bearing of the abutment. Give the dimension, from centerline of bearing to backface of abutment along the longitudinal reference line and the offset distance if on a skew. Show the skew angle.

If there is piling, show a complete footing layout giving piling dimensions tied to the reference line. Number all the piles. Give the type of piling, length and required driving resistance. Show a welded field splice for cast in place concrete or steel H piles.

Bridge seats for steel bearings and laminated elastomeric bearings are level within the limits of the bearing plate. Slope the bearing area utilizing non-laminated elastomeric bearings if the slope of the bottom of girder exceeds 1%. Slope the bridge seat between bearings 1” from front face of backwall to front face of abutment. Give all beam seat elevations.

1. Plan View

    a. Place a keyed construction joint near the center of the abutment if the length of the body wall exceeds 50 feet. Make the keyway as large as feasible and extend the horizontal bar steel through the joint.

    b. Dimension wings in a direction parallel and perpendicular to the wing centerline.

    c. Dimension angle between wing and body if that angle is different from the skew angle of the abutment.

2. Elevation

    a. Give beam seat, wing (front face and wing tip), and footing elevations to the nearest .01 of a foot.

    b. Give vertical dimension of wing.

3. Wing Elevation

4. Body Section

    Place an optional keyed construction joint in the parapet at the bridge seat elevation if there is a parapet.

5. Wing Sections

6. Bar Steel Listing and Detail

Use the following views where necessary:

7. Pile Plan & Splice Detail

8. View Showing Limits of Excavation and Backfill

9. Special Details for Utilities

10. Drainage Details

6.3.2.4 Piers

Use as many sheets as necessary to show all details clearly. Each substructure unit should have its own plan sheet(s).

Give dimensions for a skewed pier to a reference line which passes through the intersection of the longitudinal structural reference line and the pier centerline. Show the skew angle. Dimension the centerline spacing of superstructure girders.

1. Plan View

   Show dimensions, footings, cap steps, beam spacings and skew angle.

2. Elevation

   Show dimensions and elevations. Show lengths of all columns for clarity. Give the elevation of the bottom of footings and beam seats. Refer to abutments for detailing bridge seats. Dimension all bar steel and stirrups.

3. Footing Plan

   Show dimensions for pile spacing, pile numbers and reinforcing steel in footing.

4. Bar Steel Listing and Details

5. Pile Splice Detail (If different from abutment only).

6. Cross Section thru Column and Pier Cap

   Detail anchor bolts between reinforcing bars to provide clearance. Long steel bridges may require more clearance. This allows an erection tolerance for the structural steel so that the bar steel is not pierced by the anchor bolts if the bearing is shifted.

6.3.2.5 Superstructure

Use as many sheets as are necessary to show all details clearly. Standard insert sheets are available to show many standard details. The title, project number, and a few basic dimensions are added to these standard sheets.

6.3.2.5.1 All Structures

1. Show the cross-section of roadway, plan view and related details, elevation of typical girder or girders, details of girders, and other details not shown on standard insert sheets. All drawings are to be fully dimensioned and show such sections and views as needed to detail the superstructure completely.

2. For girder bridges:
   Show the total dead load deflections, including composite dead load (without future wearing surface) acting on the composite section, at tenth points of each span. Distribute the composite dead load evenly to all girders and provide one deflection value for a typical interior girder. Chapter 17 – Superstructure-General illustrates three load cases for exterior girder design with raised sidewalks, cases that provide a conservative envelope to ensure adequate girder capacity. However, the above composite dead load distribution should be used for deflection purposes. For prestressed concrete girders, the dead load deflection reported does not include the weight of the girder. See Chapter 24 – Steel Girder Structures for camber and blocking, top of steel elevation and deflection reporting criteria.

   A separate deflection value for interior and exterior girders *may* be provided if the difference, accounting for load transfer between girders, warrants multiple values. A weighted distribution of composite dead load could be used for deflection purposes *only*. For example, an extremely large composite load over the exterior girder could be distributed as 40-30-30 percent to the exterior and first two interior girders respectively. Use good engineering judgment to determine whether to provide separate deflection values for individual girder lines. In general, this is not necessary.

   For slab bridges:
   Provide camber values at the tenth points of all spans. The camber is based on 3 times the deflection of the slab, only. For multi-span bridges, the deflection calculations are based on a continuous span structure since the falsework supports the bridge until the concrete slab has cured.

   Deflection and camber values are to be reported to the nearest 0.1 inch, for all girder and slab superstructures.

3. For girder structures, provide finished grade top of deck elevations for each girder line at the tenth points of all spans. Show the top of deck elevations at the outside edge of deck at tenth points. If staged construction, include tenth point elevations along the construction joint. For slab structures, provide the finished grade elevations at the reference line and/or crown and edge of slab at tenth points.

4. Decks of uniform thickness are used on all girders. Variations in thickness are achieved by haunching the deck over each girder. Haunches are formed off the top of the top flange. See the standards for details. In general the minimum haunch depth along the edge of girder is to be 1 1/4” although 2” is recommended to allow for construction tolerances. Haunch depth is the distance from the bottom of the concrete deck to the top of the top flange.

5. Provide a paving notch at each end of all structures for rigid approach pavements. See standard for details.

6. If the structure contains conduit for a deck lighting system, place the conduit in the concrete parapet. Place expansion devices on conduit which passes through structure expansion joints.

7. Show the bar steel reinforcement in the slab, curb, and sidewalk with the transverse spacing and all bars labeled. Show the direction and amount of roadway crown.

8. On bridges with a median curb and left turn lane, water may be trapped at the curb due to the grade slope and crown slope. If this is the case, make the cross slope flat to minimize the problem. Existing pavers cannot adjust to a variable crown line.

9. On structures with modular joints consider cover plates for the back of parapets when aesthetics are a consideration.

10. Provide a table of tangent offsets for the reference line and edges of deck at 10 foot intervals for curved bridges.

6.3.2.5.2 Steel Structures

1. Show the diaphragm connections on steel girders. Show the spacing of rail posts on the plan view.

2. Show a steel framing plan for all steel girders. Show the spacing of diaphragms.

3. On the elevation view of steel girders show dimension, material required, field and shop splice locations, stiffener spacing, shear connector spacing, and any other information necessary to construct the girder. In additional views show the field splice details and any other detail that is necessary.

4. Show the size and location of all weld types with the proper symbols except for butt welds. Requirements for butt welds are covered by A.W.S. Specifications.

5. See Chapter 24 – Steel Girder Structures for camber and blocking, top of steel elevation and deflection reporting criteria.

6. Existing flange and web sizes should be shown to facilitate the sizing of bolts on Rehabilitation Plans.

6.3.2.5.3 Railing and Parapet Details

Standard drawings are maintained by the Structures Development Section showing railing and parapet details. Add the details and dimensions to these drawings that are unique to the structure being detailed. Compute the length along the slope of grade line rather than the horizontal dimension.

6.3.3 Miscellaneous Information

6.3.3.1 Bill of Bars

Show a complete bill of bar steel reinforcement for each unit of the structure. Place this bill on the sheet to which the bars pertain. If the abutments or piers are similar, only one bar list is needed for each type of unit.

Give each bar or group of bars a different mark if they vary in size, length, or location in a unit. Each bar list is to show the mark, number of bars, length, location and detail for each bar. Give bar lengths to the nearest 1" and segment lengths of bent bars to the nearest 1/2". Show all bar bends and hooks in detail.

Identify all bars with a letter indicating the unit in which the bar is placed - A for abutment, P for pier, S for superstructure. Where units are multiple, each unit should have a different letter. Next use a one or two digit number to sequentially number the bars in a unit. P1008 indicates bar number 08 is a size number 10 bar located in a Pier.

Use a Bar Series Table where a number of bars the same size and spacing vary in length is a uniform progression. Use only one mark for all these bars and put the average length in the table.

Refer to the Standard drawings in Chapter 9 – Materials for more information on reinforcing bars such as minimum bend diameter, splice lengths, bar supports, etc.

When a bridge is constructed in stages, show the bar quantities for each stage. This helps the contractor with storage and retrieval during construction.

6.3.3.2 Box Culverts

Detail plans for box culverts are to be fully dimensioned and have sectional drawings needed to detail the structure completely. The following items are to be shown when necessary:

1. Plan View
2. Longitudinal section
3. Section thru box
4. Wing elevations
5. Section thru wings
6. Section thru cutoff wall
7. Vertical construction joint
8. Bar steel clearance details
9. Header details

10. North point, Bench mark, Quantities
11. Bill of bars, Bar details
12. General notes, List of drawings, Rip rap layout
13. Inlet nose detail on multiple cell boxes
14. Corner details

Bid items commonly used are excavation, concrete masonry, bar steel, rubberized membrane waterproofing, backfill and rip rap. Filler is a non-bid item. In lieu of showing a contour map, show profile grade lines as described for Subsurface Exploration sheet.

See the standard details for box culverts for the requirements on vertical construction joints, apron and cutoff walls, longitudinal construction joints, and optional construction joints.

Show name plate location on plan view and on wing detail.

6.3.3.3 Miscellaneous Structures

Detail plans for other structures such as retaining walls, pedestrian bridges, and erosion control structures are to be detailed with the same requirements as previously mentioned. Multiple sign structure of the same type and project may be combined into a single set of plans per standard insert sheet provisions, and shall be subject to the same requirements for bridge plans.

6.3.3.4 Standard Drawings

Standard drawings are maintained and furnished by the Structures Development Section. These drawings show the common types of details required on the contract plans.

6.3.3.5 Insert Sheets

These sheets are maintained by the Structures Development Section and are used in the contract plans to show standard details.

6.3.3.6 Change Orders and Maintenance Work

These plans are drawn on full size sheets. A Structure Survey Report should be submitted for all maintenance projects, including painting projects and polymer overlay projects. In addition to the SSR, final structure plans on standard sheet borders with the #8 tab should be submitted to BOS in the same fashion as other rehabilitation plans. Painting plans should include at minimum a plan view with overall width and length dimensions, the number of spans, an indication of the number and type of elements to be painted (girders, trusses, etc.), and an elevation view showing what the structure is crossing. The SSR should give a square foot quantity for patchwork painting. For entire bridges or well defined zones (e.g. Paint all girders 5 feet on each side of expansion joints), the design engineer will be responsible for determining the quantity.

6.3.3.7 Name Plate and Bench Marks

WisDOT has discontinued the statewide practice of furnishing bench mark disks and requiring them to be placed on structures. However, WisDOT Region Offices may continue to provide bench mark disks for the contract to be set. Bench mark disks shall be shown on all bridge and larger culvert plans. Locate the bench mark disks on a horizontal surface flush with the concrete. Bench marks to be located on top of the parapet on the bridge deck, above the first right corner of the abutment traveling in the highway cardinal directions of North or East. Locate the name plate on the roadway side of the first right wing or railing traveling in the highway cardinal directions of North or East. For type “NY”, “W”, “M” or timber railings, name plate to be located on wing. For all other railing types, name plate to be located on inside face of railing.

6.3.4 Checking Plans

Upon completion of the design and drafting of plans for a structure, the final plans are usually checked by one person. Dividing plans checking between two or more Checkers for any one structure leads to errors many times. The plans are checked for compliance with the approved preliminary drawing, design, sufficiency and accuracy of details, dimensions, elevations, and quantities. Generally the information shown on the preliminary plan is to be used on the final plans. Revisions may be made to footing sizes and elevations, pile lengths, dimensions, girder spacing, column shapes, and other details not determined at the preliminary stage. Any major changes from the preliminary plan are to be approved by the Structural Design Engineer and Supervisor.

The Checkers check the final plans against the Engineer's design and sketches to ensure all information is shown correctly. The Engineer prepares all sketches and notations not covered by standard drawings. A good Checker checks what is shown and noted on the plan and also checks to see if any essential details, dimensions, or notation have been omitted. The final plan Bid Items should be checked for conformity with those listed in the WisDOT Standard Specifications for Highway and Structure Construction.

The Checker makes an independent check of the Bill of Bars list to ensure the Plan Preparer has not omitted any bars when determining the quantity of bar steel.

Avoid making minor revisions in details or dimensions that have very little effect on cost, appearance, or adequacy of the completed structure. Check grade and bridge seat elevations and all dimensions to the required tolerances. The Checkers make all corrections, revisions, and notations on a print of the plan and return it to the Plan Preparer. The Plan Preparer back checks all marks made by the Checker before changing. Any disagreements are resolved with the Supervisor.

Common complaints received from field staff are dimension errors, small details crowded on a drawing, lettering is too small, and reinforcing bar length or quantity errors.

After the plans are completed, the items in the project folder are separated into the following groups by the Structures Design Engineer:

6.3.4.1 Items requiring a PDF copy for the Project Records (Group A) – Paper Copies to be Destroyed when Construction is Completed.

1. QC/QA sign-off sheet
2. Design computations and computer runs
3. Quantity computations
4. Bridge Special Provisions and STSP's (only those STSP's requiring specific blanks to be filled in or contain project specific information)
5. Final Structure Survey Report form (not including photos, cross-sections, project location maps, etc.)
6. Final Geotechnical Report
7. Final Hydrology and Hydraulic computations and structure sizing report
8. Contour map

6.3.4.2 Additional Items to be Destroyed When Construction is Completed (Group B)

1. Miscellaneous correspondence and transmittal letters
2. Preliminary drawings and computations
3. Prints of soil borings and plan profile sheets
4. Shop steel quantity computations*
5. Design checker computations
6. Layout sheets
7. Elevation runs and bridge geometrics
8. Falsework plans*
9. Miscellaneous Test Report
10. Photographs of bridge rehabs

* These items are added to the packet during construction.

6.3.4.3 Items to be Destroyed when Plans are Completed (Group C)

1. All "void" material
2. All copies except one of preliminary drawings

3. Extra copies of plan and profile sheets

4. Preliminary computer design runs

Note that lists for Group A, B & C are not intended to be all inclusive, but serve as starting points for categorizing design material. Items in Group A & B should be labeled separately. Computation of Quantities

6.4 Computation of Quantities

When the final drafting and plan checking is completed, the person responsible for drafting the plans and plans checker are to prepare individual quantity calculations for the bid items listed on the plans. The following instructions apply to the computation on quantities.

Divide the work into units that are repetitive such as footings, columns, and girders. Label all items with a clear description. Use sketches for clarity. These computations may be examined by others in future years so make them understandable.

One of the most common errors made in quantity computation is computing only half of an item which is symmetrical about a centerline and forgetting to double the result.

Following is a list of commonly used bridge quantities. Be sure to use the appropriate item and avoid using incidental items as this is too confusing for the contractor and project manager. The bid item for Abatement of Asbestos Containing Material should be included on the structure plans. Items such as Incentive Strength Concrete Structures, Construction Staking Structure Layout, etc. should not be included on the structure plans.

A column with the title “Bid Item Number” should be the first column for the “Total Estimated Quantities” table shown in the plans. The numbers in this column will be the numbers associated with the bid items as found in the Standard Specification, STSP, and/or Special Provisions.

6.4.1 Excavation for Structures Bridges (Structure)

This is a lump sum bid item. The limits of excavation are shown in the chapter in the manual which pertains to the structural item, abutments, piers, retaining walls, box culverts, etc. If the excavation is required for the roadway, the work may be covered under Excavation Common.

The limits of excavation made into solid rock are the neat line of the footing.

6.4.2 Backfill Granular or Backfill Structure

Backfill Granular and Backfill Structure are bid in units of cubic yard. The pay limits and quantity computations of backfill at abutments are shown in Chapter 12 – Abutments.

6.4.3 Concrete Masonry Bridges

Show unit quantities to the nearest cubic yard, as well as the total quantity. In computing quantities no deduction is made for metal reinforcement, floor drains, conduits and chamfers less than 2”. Flanges of steel and prestressed girders projecting into the slab are deducted.

Deduct the volume of pile heads into footings and through seals for all piling except steel H sections. Deduct the actual volume displaced for precast concrete and cast-in-place concrete piling.

Consider the concrete parapet railing on abutment wing walls as part of the concrete volume of the abutment.

6.4.4 Prestressed Girder Type I (28-Inch; 36-Inch; 36W-Inch; 45W-Inch; 54W-Inch; 72W-Inch, 82W-Inch)

Record the total length of prestressed girders to the nearest 1 foot.

6.4.5 Bar Steel Reinforcement HS Bridges or Bar Steel Reinforcement HS Coated Bridges

Record this quantity to the nearest 10 lbs. Designate if bar steel is coated. Include the bar steel in C.I.P. concrete piling in bar steel quantities.

6.4.6 Bar Steel Reinforcement HS Stainless Bridges

Record this quantity to the nearest 10 lbs. Bar weight shall be assumed to be equivalent to Bar Steel Reinforcement HS Bridges or Bar Steel Reinforcement HS Coated Bridges bid items.

6.4.7 Structural Steel Carbon or Structural Steel HS

See 24.2.4.

6.4.8 Bearing Pads Elastomeric Non-Laminated or Bearing Pads Elastomeric Laminated or Bearing Assemblies Fixed (Structure) or Bearing Assemblies Expansion (Structure)

Record as separate item with quantity required. Bid as Each.

6.4.9 Piling Test Treated Timber (Structure)

Record this quantity as a lump sum item. Estimate the pile lengths by examining the subsurface exploration sheet and the Site Investigation Report. Give the length and location of test piles in a footnote. Do not use this quantity for steel piling or concrete cast-in-place piling.

6.4.10 Piling CIP Concrete Delivered and Driven ___-Inch, Piling Steel Delivered and Driven ___ -Inch

Record this quantity in feet for Steel and C.I.P. types of piling delivered and driven. Timber piling are Bid as separate items, delivered and driven. Pile lengths are computed to the nearest 5.0 foot for each pile within a given substructure unit, unless a more exact length is known due to well defined shallow rock (approx. 20 ft.), etc.. Typically, all piles within a given substructure unit are shown as the same length.

The length of foundation piling driven includes the length through any seal and embedment into the footing. The quantity delivered is the same as quantity driven. For trestle piling the amount of piling driven is the penetration below ground surface.

Oil field pipe is allowed as an alternate on all plans unless a note is added in the General Notes stating it is not allowed on that specific project.

6.4.11 Preboring CIP Concrete Piling or Steel Piling

Record the type, quantity in feet. Calculate to the nearest lineal foot per pile location.

6.4.12 Railing Steel Type (Structure) or Railing Tubular Type (Structure)

Record the type and quantity, bid in lineal feet. For bridges, the railing length should be horizontal length shown on the plans. For retaining walls, use the length along the top of the wall. Calculate railing lengths as follows:

- Steel Railing Type 'W' – CL end post to CL end post
- Tubular Railing Type 'H' – CL end plate to CL end plate
- Combination Railing Type '3T' – CL end post to CL end post + (2'-5") per railing
- Tubular Railing Type 'M' – CL end post to CL end post + (4'-6") per railing
- Combination Railing Type 'Type C1-C6' – CL end rail base plate to CL end rail base plate
- Tubular Steel Railing Type NY3&4 – CL end post to CL end post + (4'-10") per railing

6.4.13 Slope Paving Concrete or Slope Paving Crushed Aggregate or Slope Paving Select Crushed Material

Record this quantity to the nearest square yard. Deduct the area occupied by columns or other elements of substructure units.

6.4.14 Riprap Medium, Riprap Heavy or Grouted Riprap, Riprap Light

Record this quantity to the nearest 5 cubic yards.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

6.4.16 Floordrains Type GC, Floordrains Type H, or Floordrains Type WF

Record the type and number of drains. Bid as Each.

6.4.17 Cofferdams (Structure)

Lump Sum

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

For "Expansion Device" and "Expansion Device Modular", bid the items in lineal feet. The distance measured is from flowline to flowline along the skew (do not include turn-ups into parapets or medians).

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

6.4.21 Conduit Rigid Metallic __-Inch or Conduit Rigid Nonmetallic Schedule 40 -Inch

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

Record these quantities to the nearest square yard. Type 1 Deck Preparation should be provided by the Region. Estimate Type 2 Deck Preparation as 40% of Type 1 Deck Preparation. Deck preparation areas shall be filled using Concrete Masonry Overlay Decks, Concrete Masonry Deck Patching, or with an appropriate deck patch. See Chapter 40 Standards.

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

Record this quantity to the nearest cubic yard. Estimate the quantity by using a thickness measured from the existing ground concrete surface to the plan gradeline. Calculate the minimum overlay thickness and add ½" for variations in the deck surface. Provide this average thickness on the plan, as well. Usually 1" of deck surface is removed by grinding. Include deck repair quantities for Preparation Decks Type 1 & 2 and Full-Depth Deck Repair. Use 2-inch thickness for each Preparation area and ½ the deck thickness for Full-Depth Deck Repairs in areas of deck preparation (full-depth minus grinding if no deck preparation).

6.4.28 Removing Old Structure STA. XX + XX.XX

Covers the entire or partial removal of an existing structure. Bid as Lump Sum.

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

Attachment assembly for Beam Guard at the termination of concrete parapets. Bid as each.

6.4.30 Steel Diaphragms (Structure)

In span diaphragms used on bridges with prestressed girders. Bid as each.

6.4.31 Welded Stud Shear Connectors X -Inch

Total number of shear connectors with the given diameter. Bid as each.

6.4.32 Concrete Masonry Seal

Seal concrete bid to the nearest cubic yard. Whenever a concrete seal is shown on the plans, then "Cofferdams (Structure)" is also to be a bid item.

6.4.33 Geotextile Fabric Type

List type of fabric. Type HR is used in conjunction with Heavy Riprap. Bid in square yards.

6.4.34 Masonry Anchors Type L No. Bars

Used when anchoring reinforcing bars into concrete. Bid as each.

6.4.35 Piling Steel Sheet Permanent Delivered or Piling Steel Sheet Permanent Driven

Record this quantity to the nearest square foot for the area of wall below cutoff.

6.4.36 Piling Steel Sheet Temporary

This quantity is used when the designer determines that retention of earth is necessary during excavation and soil forces require the design of steel sheet piling.

Record this quantity to the nearest square foot for the area below the retained grade and one foot above the retained grade.

Following is a list of commonly used STSP's and Bureau of Structures Special Provisions.

6.4.37 Temporary Shoring

This quantity is used when earth retention may be required and the method chosen is the contractor's option.

Bid as square foot of exposed surface as shown on the plans.

6.4.38 Concrete Masonry Deck Patching

Record this quantity to the nearest cubic yard. Use 2-inch thickness for each Preparation area and ½ the deck thickness for Full-Depth Deck Repairs.

6.4.39 Sawing Pavement Deck Preparation Areas

Use 10 lineal feet per SY of Preparation Decks.

6.4.40 Removing Bearings

Used to remove existing bearings for replacement with new expansion or fixed bearing assemblies. Bid as each.

6.4.41 Ice Hot Weather Concreting

Used to provide a mechanism for payment of ice during hot weather concreting operations. See FDM 19-7-1.2 for bid item usage guidance and quantity calculation guidance. Bid as LB and round to the nearest 5 lbs.

6.5 Production of Bridge Plans by Consultants, Regional Offices and Other Agencies

The need for structures is determined during the Preliminary Site Survey and recorded in the Concept Definition or Work Study Report. On Federal (FHWA) or State Aid Projects completed Structure Survey Reports and plans are submitted to the Bureau of Structures with a copy forwarded to the Regional Office for approval prior to construction. Structure and project numbers are provided by the Regional Offices. In preparation of the structural plans, the appropriate specifications and details recommended by the Bureau of Structures are to be used. If the consultant elects to modify or use details other than recommended, approval is required prior to their incorporation into the final plans.

On all Federal or State Aid Projects involving Maintenance work, the Concept Definition or Work Study Report, the preliminary and final bridge reconstruction plans shall be submitted to the Bureau of Structures for review.

Consultants desiring eligibility to perform engineering and related services on WisDOT administered structure projects must have on file with the Bureau of Structures, an electronic copy of their current Quality Assurance/Quality Control (QA/QC) plan and procedures. The QA/QC plan and procedures shall include as a minimum:

- Procedures to detect and correct bridge design errors before the design plans are made final.
- A means for verifying that the appropriate design calculations have been performed, that the calculations are accurate, and that the capacity of the load-carrying members is adequate with regard to the expected service loads of the structure.
- A means for verifying the completeness, constructability and accuracy of the structure plans.
- Verification that independent checks, reviews and ratings were performed.

A QA/QC verification summary sheet is required as part of every final structure plan submittal, demonstrating that the QA/QC plan and procedures were followed for that structure. The QA/QC verification summary sheet shall include the signoff or initialing by each individual that performed the tasks (design, checking, plan review, technical review, etc.) documented in the QA/QC plan and procedures. The summary sheet must be submitted with the final structure plans as part of the e-submit process.

6.5.1 Approvals, Distribution, and Work Flow

Consultant	Meet with Regional Office and/or local units of government to determine need.
	Prepare Structure Survey Report including recommendation of structure type.
Geotechnical Consultant	Make site investigation and prepare Site Investigation Report.
Consultant	Prepare Preliminary Plan documents including scour computations for spread footings and/or shallow pile foundations. Record scour critical

	code on preliminary plans. Refer to Chapter 8, Appendix 8-D.
	If a navigable waterway is crossed, complete necessary Coast Guard coordination.
	Submit preliminary plans and documents via e-submit for review and approval of type, size and location.
Structures Design Section	Record Bridge and project numbers.
	Review hydraulics for Stream Crossings.
	Review Preliminary Plan. A minimum of 30 days to review preliminary plans should be expected.
	For special structure types (lift or moveable bridges; cost greater than $10,000,000), send preliminary plans to Federal Highway Administration for approval.
	Return preliminary plans and comments from Structures Design Section and other appropriate agencies to Consultant with a copy to the Regional Office.
	Forward Preliminary Plan and Hydraulic Data to DNR.
Consultant	Modify preliminary plan as required.
	Prepare and complete final design and plans for the specified structure.
	Write special provisions.
	At least **two months** in advance of the PS&E date, submit the following via e-submit: sealed, signed and dated final plans, special provisions, computations, quantities, QA/QC Verification Sheet, Inventory Data Sheet, Bridge Load Rating Summary Form, LRFD Input File (Excel ratings spreadsheet).
Structures Design Section	Determine which final plans will be reviewed and perform review as applicable.
	For special structure types (lift or moveable bridges; cost greater than $10,000,000), send final plans to Federal Highway Administration.
	For final plans that are reviewed, return comments to Consultant and send copy to Regional Office.
Consultant	Modify final plans and specifications as required.
	Submit modified final plans via e-submit as required.
Structures Design Section	Review modified final plans as applicable.
	Sign final plans.

Bureau of Project Development	Prepare final accepted structure plans for pre-development contract administration.

Table 6.5-1
Approvals, Distribution and Work Flow

6.5.2 Consultant Preliminary Plan Requirements

The Consultant prepares the Structure Survey Report for the improvement. Three types of Structure Survey Reports are available at the Regional Offices and listed in 6.2.1 of this Chapter. Preliminary layout requirements are given in 6.2.2. The Preliminary Plan exhibits are as follows:

1. Structure Survey Report.
2. Preliminary Drawings.
3. Log Borings shown on the Subsurface Exploration Drawing which must be submitted now and can be included with the Final Plans.
4. Evaluation Report of Borings.
5. Contour Map.
6. Typical Section for Roadway Approaches.
7. Plan and Profile of Approach Roadways.
8. Hydraulic/Sizing Report (see Chapter 8 - Hydraulics) is required for Stream Crossing Structures.
9. County Map showing Location of New and/or Existing Structures.
10. Any other information or Drawings which may influence Location, Layout or Design of Structure.

The above information is also required for Box Culverts. Bureau of Structures personnel will review the structure type and may recommend that other types be considered. In this regard it is extremely important that preliminary designs be coordinated to avoid delays and unnecessary expense in plan preparation.

If the final approach roadways are unpaved, detail protective armor angles at the roadway ends of bridge decks/slabs as shown on the Standard for Strip Seal Cover Plate Details.

6.5.3 Final Plan Requirements

The guidelines and requirements for Final Plan preparation are given in 6.3. The following exhibits are included as part of the Final Plans:

1. Final Drawings

   For all highway structures provide the maximum vehicle weight that can be safely carried based on the procedure and vehicle configuration provided in Chapter - Bridge Rating.

2. Design and Quantity Computations

   For all bridge structures, provide the analysis files used by the responsible engineer for determination of the controlling ratings.

3. Special Provisions covering unique items not in the Standard Specifications such as Electrical Equipment, New Proprietary Products, etc.

4. QA/QC Verification Sheet

5. Inventory Data Sheet, Bridge Load Rating Summary Form, LRFD Input File (Excel ratings spreadsheet).

On Federal or State Aid projects the contracts are let and awarded by the Wisconsin Department of Transportation. Shop drawing review and fabrication inspection are generally done by the Metals Fabrication and Inspection Unit. However, in some cases the consultant may check the shop drawings and an outside agency may inspect the fabrication. The Consultant contract specifies the scope of the work to be performed by the Consultant. Construction supervision and final acceptance of the project are provided by the State.

6.6 Structures Data Management and Resources

6.6.1 Structures Data Management

The following items are part of the Data Management System for Structures. The location is shown for all items that need to be completed in order to properly manage the Structure data either by Structures Design personnel for in-house projects or consultants for their designs.

1. Structure Survey Report - Report is submitted by Region or Consultant and placed in the individual structure folder in HSI by BOS support staff.

2. Subsurface Exploration Report - Report is submitted by WisDOT Geotechnical Engineering Unit and placed in the individual structure folder in HSI by BOS support staff.

3. Hydraulic and Scour Computations, Contour Maps and Site Report - Data is assembled by the BOS Consultant Review & Hydraulics Unit and placed in the individual structure folder in HSI by BOS support staff.

4. Designer Computations and Inventory Superstructure Design Run (Substructure computer runs as determined by the Engineer). The designers record design, inventory, operating ratings and maximum vehicle weights on the plans.

5. Load Rating Input File and Load Rating Summary sheet - The designer submits an electronic copy of the input data for load rating the structure to the Structures Development Section. (For internal use, it is located at //H32751/rating.)

6. Structure Inventory Form (Available under “Inventory & Rating Forms” on the HSI page of the BOS website). New structure or rehabilitation structure data for this form is completed by the Structural Design Engineer. It is E-submitted to the Structures Development Section for entry into the File.

7. Pile Driving Reports - An electronic copy of Forms DT1924 (Pile Driving Data) and DT1315 (Piling Record) are to be submitted to the Bureau of Structures by email to “DOTDTSDStructuresPiling@dot.wi.gov ”. These two documents will be placed in HSI for each structure and can be found in the “Shop” folder.

8. Final Shop Drawings for steel bridges (highway and pedestrian), sign bridges, floor drains, railings, all steel joints, all bearings, high-mast poles, prestressed girders, prestressed boxes, noise walls and retaining walls. Metals Fabrication & Inspection Unit or others submit via email to the Structures Development Section at “DOTDLDTSDSTRUCTURESRECORDS@DOT.WI.GOV ”. This process does not, however, supersede submission processes in place for specific projects.

9. Mill Tests, Heat Numbers and Shop Inspection Reports for all Steel Main Members. Metals Fabrication & Inspection Unit electronically files data into HSI

10. As-Let Plans: After bid letting, a digital image of the As-Let plans are placed in a computer folder in the Bureau of Project Development (BPD). BOS office support

staff extract a digital copy of the As-Let structure plans and place it in the structure folder for viewing on HSI.

11. As-Built Plans: As-Built structure plans shall be prepared for all let structure projects, new or rehabilitation. The structures with prefix ‘B’, ‘P’, ‘C’, ‘M’, ‘N’, ‘R’ and ‘S’ shall have As-Built plans produced after construction. The As-Built shall be prepared in accordance with Section 1.65.14 of the Construction and Materials Manual (CMM). These plans are located on a network drive and be viewed in DOTView GIS. BOS staff will ensure that the proper BOS folder (\\dotstrc\04bridge) has a copy of these plans for viewing in HSI.

12. Inspection Reports - A certified bridge inspector enters the initial and subsequent inspection data into HSI.

Initial	Underwater (UW-Probe/Visual)
Routine Visual	Movable
Fracture Critical	Damage
In-Depth	Interim
Underwater (UW)-Dive	Posted
Underwater (UW)-Survey	Structure Inventory and Appraisal

Table 6.6-1
Various Inspection Reports

** HSI – Highway Structures Information System – The electronic file where bridge data is stored for future use.

6.6.2 Resources

The following items are available for assistance in the preparation of structure plans on the department internet sites:

http://on.dot.wi.gov/dtid_bos/extranet/structures/LRFD/index.htm

Bridge Manual
Highway Structures Information System (HSI)
Insert sheets
Standard details
Posted bridge map
Standard bridge CADD files
Structure survey reports and check lists
Structure costs
Structure Special Provisions

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/manuals.aspx

Facilities Development Manual
Standard Specifications for Highway and Structures Construction
Construction and Materials Manual

Additional information is available on the AASHTO and AREMA websites listed below:

http://bridges.transportation.org

https://www.arema.org/

Table of Contents

8.1 Introduction

The methods of hydrologic and hydraulic analysis provided in this chapter give the designer information necessary for an analysis of a roadway drainage crossing. Experience and sound engineering judgment are not to be ignored and may, at times, differ from results obtained using methods in this chapter. Very careful weighing of experience, judgment, and procedure must be made to arrive at a solution to the problem. Research in the field of drainage continues throughout the country and may subsequently alter the procedures found in this chapter.

8.1.1 Objectives of Highway Drainage

The objective of highway drainage is to prevent the accumulation and retention of water on and/or around the highway by:

- Anticipating the amount and frequency of storm runoff.
- Determining natural points of concentration of discharge and other hydraulic controls.
- Removing detrimental amounts of surface and subsurface water.
- Providing the most efficient hydraulic design consistent with economy, the importance of the road, maintenance and legal obligations.

8.1.2 Basic Policy

In designing highway drainage, there are three major considerations; first, the safety of the traveling public, second, the design should be in accordance with sound engineering practices to economically protect and drain the highway, and third, in accordance with reasonable interpretation of the law, to protect private property from flooding, water soaking or other damage. In general, the hydraulic adequacy of structures is determined by the methods as outlined in this manual and performance records of structures in the same or similar locations.

8.1.3 Design Frequency

Federal and State governments have placed increasing emphasis on environmental protection over the last several years. Consequently the administrative rules established by regulatory agencies have made past practice of designing structures to accommodate flood frequencies of 25 and 50 years obsolete and unworkable. Thus, the design discharge for all bridges and box culverts covered under this chapter shall be the 100 year (Q_{100}) frequency flood. In floodplain management this is also referred to as the Regional or Base flood. Design frequency is determined from requirements in Federal Highway Administration (FHWA) directives and the co-operative agreement between Wisconsin Department of Transportation (DOT) and Wisconsin Department of Natural Resources (DNR). The following publications are suggested for guidance.

8.1.3.1 FHWA Directive

Title 23, Chapter 1, Sub Chapter G, Part 650, Subpart A of the FHWA – Federal-Aid Policy Guide, "*Location and Hydraulic Design of Encroachments on Flood Plains*", prescribes FHWA policy and procedures. Copies of this directive may be found on the FHWA website.

8.1.3.2 DNR-DOT Cooperative Agreement

The Wisconsin Department of Transportation and the Wisconsin Department of Natural Resources have signed a co-operative agreement to provide a reasonable and economical procedure for carrying out their respective duties in a manner that is in the total public interest. The provisions in this agreement establish the basic considerations for highway stream crossings. A copy of this agreement can be found in Figure 1, Procedure 20-30-1 of the Wisconsin DOT Facilities Development Manual (FDM).

8.1.3.3 DOT Facilities Development Manual

Refer to Procedures - Chapter 13 - Drainage Practice, Chapter 20 - Environmental Laws, Policies and Regulations and Chapter 21 - Environmental Documents, Reports and Permits.

8.1.4 Hydraulic Site Report

A hydraulic report for all projects shall be submitted with the "Stream Crossings Structure Survey Report" for Bridges and Box Culverts. A check list of the various discussion items that need to be provided in the hydraulic site report is included as 8.6 Appendix 8-A. Plan survey datum must conform to datum in use by local zoning authorities. In most cases elevations are referenced to the National Geodetic Vertical Datum (NGVD) of 1929, or to the North American Vertical Datum of 1988 (NAVD 88). The Hydraulic Site Report discusses and documents the hydrologic, hydraulic, site conditions, and all other pertinent factors that influence the type, size, and location of the proposed structure.

8.1.5 Hydraulic Design Criteria for Temporary Structures

The basic design criteria for temporary structures will to be the ability to pass a 5-year storm (Q5) with only 0.5 feet of backwater over existing conditions. This criteria is only a general guideline and site specific factors and engineering judgment may indicate that this criteria is inappropriate. Separate hydraulic design criteria should be used for the design of temporary construction causeways. Factors that should be considered in the design of temporary structures and approach embankments are:

- Effects on surrounding property and buildings
- Velocities that would cause excessive scour
- Damage or inconvenience due to failure of temporary structure
- DNR concerns

- Temporary roadway profile
- Structure depths will be 36” for short spans and 48” or more for longer spans.

If possible and practical, the temporary roadway profile should be designed and constructed in such a manner that infrequent flood events are not obstructed from overflowing the temporary profile and creating excessive backwaters upstream of the construction. The temporary roadway profile should provide adequate clearance for the temporary structure.

The roadway designer should indicate the need for a temporary structure on the Stream Crossing Structure Survey Report. Preliminary and Final plans should indicate the hydraulic parameters of the temporary structure. The required parameters are the 5-year flood discharge (Q5), the 5-year high-water elevation (HW5), and the flow area of the temporary structure required to pass the 5-year flood (Abr).

8.1.6 Erosion Control Parameters

In order to assist designers in determining the appropriate erosion control measures to be provided at Bridge construction site, preliminary and final plans should indicate the 2-year flood discharge (Q2) and the 2-year high-water elevation (HW2).

8.1.7 Bridge Rehabilitation and Hydraulic Studies

Generally no hydraulic study will be required in bridge rehabilitation projects that do not involve encroachment to the Base Floodplain. This includes entire super structure replacement provided that the substructure and berm configuration remain unchanged and the low cord elevation is not significantly lowered.

The designer should consider historical high-water elevations, Flood Insurance Studies and the potential of inundation when choosing the replacement superstructure type. The risk of damage to the structure as the result of Scour should also be considered.

8.2 Hydrologic Analysis

The first step in designing a hydraulic structure is to determine the design discharge for the waterway. The problem is particularly difficult for small watersheds, say under five square miles, because the smaller the area, the more sensitive it is to conditions which affect runoff and the less likely there are runoff records for the area.

Acceptable methods of determining the design discharge for the 100 year flood shall be based on the guidelines contained in the *State Administrative Code NR 116.07, Wisconsin's Floodplain Management Program*[1]. Generally, a minimum of two methods should be used in determining a design discharge.

The most frequently used methods for determining the design discharge for bridges and box culverts in the State of Wisconsin are discussed below.

8.2.1 Regional Regression Equations

The U. S. Geological Survey (USGS) in cooperation with the Wisconsin Department of Transportation prepared a report entitled *Flood Frequency Characteristics of Wisconsin Streams*[2] which considers the flood potentials for a site using regional regression equations based on flood data from gaging stations on Wisconsin's rivers and streams. The flood-frequency regression equations are correlated with three or more of seven parameters, namely, drainage area, main-channel slope, storage, forest cover, mean annual snowfall, precipitation intensity index, and soil permeability. These equations are applicable to all drainage areas in Wisconsin except for highly regulated streams, and highly urbanized areas of the state.

8.2.2 Watershed Comparison

The results obtained from the above regression equations should be compared to similar gaged watersheds listed in reference (2) above using the area transfer formulas and procedures detailed in that document. A good discussion and examples of the use of regression equations and basin comparison methods can be seen in the WisDOT Facilities Development Manual, Procedure 13-10-5. The flood frequency discharges listed in reference (2) are for flood records up to the year 2000. More years of data are available from the USGS for most of the gaged watersheds.

The flood frequency discharges for the gaged watersheds can be updated past water year 2000 by using the Log-Pearson Type III distribution method as described in *Bulletin #17B entitled Guidelines For Determining Flood Flow Frequency*[3] and the guidelines for weighting the station skew with the generalized skew in *NR116.07, Wisconsin's Floodplain Management Program*[1].

8.2.3 Flood Insurance and Floodplain Studies

The Federal Emergency Management Agency (FEMA) had contracted for detailed flood studies throughout Wisconsin. They were developed for floodplain management and flood insurance purposes. These Flood Insurance Studies (FIS) which are on file with Floodplain-

Shoreland Management Section of the Wisconsin Dept. of Natural Resources (DNR) contain discharge values for many sites. These studies, along with other various floodplain studies, may be obtained from the DNR's Floodplain Analysis Interactive Map by using the following link:

http://dnr.wi.gov/topic/floodplains/mapindex.html

8.2.4 Natural Resources Conservation Service

For small watersheds in urban and rural areas, the National Resources Conservation Service (NRCS) has developed procedures to calculate storm runoff volumes, peak rates of discharge, hydrographs and storage volumes. The procedure is documented in *Technical Release 55 Urban Hydrology for Small Watersheds*[4].

8.5 References

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19. U.S. Department of Interior, Bureau of Reclamation, *Design of Small Dam*, 3rd Edition Washington D.C. 1987.

20. U.S. Department of Transportation, Federal Highway Administration, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular (HEC) No. 14, Third Edition, Publication No. FHWA-NHI-06-086, July 2006.

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9.1 General

The *Wisconsin Standard Specifications for Highway and Structure Construction* (hereafter referred to as *Standard Specifications*) contains references to *ASTM Specifications* or *AASHTO Material Specifications* which provide required properties and testing standards for materials used in highway structures. The service life of a structure is dependent upon the quality of the materials used in its construction as well as the method of construction. This chapter highlights applications of materials for highway structures and their properties.

In cases where proprietary products are experimentally specified, special provisions are written which provide material properties and installation procedures. Manufacturer's recommendations for materials, preparation and their assistance during installation may also be specified.

Materials that are proposed for incorporation into highway structure projects performed under the jurisdiction of the Wisconsin Department of Transportation (WisDOT) may be approved or accepted by a variety of procedures:

- Laboratory testing of materials that are submitted or samples randomly selected.
- Inspection and/or testing at the source of fabrication or manufacture.
- Inspection and/or testing in the field by WisDOT regional personnel.
- Manufacturer's certificate of compliance and/or manufacturer's certified report of test or analysis, either as sole documentation for acceptance or as supplemental documentation.
- Inspection, evaluation and testing in the normal course of project administration of material specifications.
- Some products are on approved lists or from fabricators, manufacturers, and certified sources approved by WisDOT. Lists of approved suppliers, products, and certified sources are located at:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

The *Wisconsin Construction and Materials Manual* (CMM) contains a description of procedures for material testing and acceptance requirements in Chapter 8, Section 45. Materials, unless otherwise permitted by the specifications, cannot be incorporated in the project until tested and approved by the proper authority.

9.2 Concrete

Concrete is used in many highway structures throughout Wisconsin. Some structure types are composed entirely of concrete, while others have concrete members. Different concrete compressive strengths (f'_c) are used in design and depend on the structure type or the location of the member. Compressive strengths are verified by cylinder tests done on concrete samples taken in the field. The *Standard Specifications* describe the requirements for concrete in Section 501.

Some of the concrete structure types/members and their design strengths for new projects are:

- Decks, Diaphragms, Overlays, Curbs, Parapets, Medians, Sidewalks and Concrete Slab Bridges (f'_c = 4 ksi)
- Other cast-in-place structures such as Culverts, Cantilever Retaining Walls and Substructure units (f'_c = 3.5 ksi)
- Other types of Retaining Walls (f'_c - values as specified in Chapter 14)
- Prestressed "I" Girders (f'_c = 6 to 8 ksi)
- Prestressed Box Girders (f'_c = 5 ksi)
- Prestressed Deck Panels (f'_c = 6 ksi)

Grade "E" concrete (Low Slump Concrete) is used in overlays for decks and slabs as stated in Section 509.2.

The modulus of elasticity of concrete, E_c, is a function of the unit weight of concrete and its compressive strength **LRFD [C5.4.2.4]**. For a unit weight of 0.150 kcf, the modulus of elasticity is:

$f'_c = 3.5$ ksi ; $E_c = 3600$ ksi

$f'_c = 4$ ksi ; $E_c = 3800$ ksi

For prestressed concrete members, the value for E_c is based on studies in the field and is calculated as shown in 19.3.3.8.

The modulus of rupture for concrete, f_r, is a function of the concrete strength and is described in **LRFD [5.4.2.6]**. The coefficient of thermal expansion for normal weight concrete is 6×10^{-6} in/in/°F per **LRFD [5.4.2.2]**.

Air entraining admixture is added to concrete to provide durability for exposure to freeze and thaw conditions. Other concrete admixtures used are set retarding and water reducing admixtures. These are covered in Section 501 of the *Standard Specifications.*

9.3 Reinforcement Bars

Reinforced concrete structures and concrete members are designed using Grade 60 deformed bar steel with a minimum yield strength of 60 ksi. The modulus of elasticity, E_s , for steel reinforcing is 29,000 ksi. Reinforcement may be epoxy coated and this is determined by its location in the structure as described below. Adequate concrete cover and epoxy coating of reinforcement contribute to the durability of the reinforced concrete structure. The *Standard Specifications* describe the requirements for steel reinforcement and epoxy coating in Section 505.

Epoxy coated bars shall be used for both top and bottom reinforcement on all new decks, deck replacements, concrete slab superstructures, structural approach slabs and top slab of culverts (with no fill on top). They shall be used in other superstructure elements such as curbs, parapets, medians, sidewalks, diaphragms and pilasters. Some of the bars in prestressed girders are epoxy coated and are specified in the Chapter 19 - Standards. Also use coated bars for sign bridge footings.

Use epoxy coated bar steel on all piers detailed with expansion joints and on all piers at grade separations. Use epoxy coated bars down to the top of the footing elevation.

At all abutments, epoxy coated bars shall be used for parapets on wing walls. For A3/A4 abutments use epoxy coated bars for the paving block and the abutment backwall, and for A1(fixed) coat the dowel bars. For all abutments use epoxy coated bars in the wing walls.

Welding of bar steel is not permitted unless approved by the Bureau of Structures or used in an approved butt splice as stated in Section 505.3.3.3 of the *Standard Specifications*. Test results indicate that the fatigue life of steel reinforcement is reduced by welding to them. Supporting a deck joint by welding attachments to the bar steel is not permitted. The bar steel mat does not provide adequate stiffness to support deck joints or similar details during the deck pour and maintain the proper joint elevations.

The minimum and maximum spacing of reinforcement, and spacing between bar layers is provided in **LRFD [5.10.3.1, 5.10.3.2]**. Use minimum and maximum values shown on Standards where provided.

Bridge plans show the quantity of bar steel required for the structure. Details are not provided for bar chairs or other devices necessary to support the reinforcement during the placement of the concrete. This information is covered by the *Standard Specifications* in Section 505.3.4 and these devices are part of the bid quantity.

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete as stated in **LRFD [5.10.8]**.

When determining the anchorage requirements for bars, consider the bar size, the development length for straight bars and the development length for standard hooks. Note in Table 9.9-1 and Table 9.9-2 that smaller bars require considerably less development length than larger bars and the development length is also less if the bar spacing is 6 inches or more. By detailing smaller bars to get the required area and providing a spacing of 6 inches or more, less steel is used. Bar hooks can reduce the required bar development lengths,

9.4 Steel

Structural steel is used in highway structures throughout Wisconsin. It is used for steel plate I-girders, rolled I-girders and box girders. Steel used for these three superstructure types are typically ASTM A709 Grades 36, 50 and 50W, but may also include high performance steel (HPS). Information on materials used for these superstructure types is provided in 24.2. Other types of steel superstructures are trusses, tied arches and cable-stayed bridges.

Steel is also used in other parts of the structure, such as:

- Bearings (Type A, B, A-T and top/interior plates for Laminated Elastomeric Bearings)
- Piling (H-Piles and CIP-Pile shells)
- Expansion Devices (single strip seal or modular joint)
- Drains (frame, grate and bracket)
- Railings (Type W, H, NY, M, PF, Tubular Screening, Fencing and Combination Railing)
- Steel diaphragms (attached to prestressed girders)

Structural carbon steel (ASTM A709 Grade 36) is used in components that are part of railings, laminated elastomeric bearings and for steel diaphragms attached to prestressed girders. Structural carbon steel (ASTM A36) is used in components that are part of drains. The minimum yield strength is 36 ksi.

High strength structural steel (ASTM A709 Grade 50) is used in H-piles and components that are part of railings and laminated elastomeric bearings. High strength structural weathering steel (ASTM A709 Grade 50W) is used in bearings. The minimum yield strength is 50 ksi.

Structural steel tubing (ASTM A500 Grades B,C) is used in components that are part of railings, such as posts or rail members. The minimum yield strengths will have values around 46 to 50 ksi.

Steel pipe pile material (ASTM A252 Grade 2) is used as the shell to form cast-in-place (CIP) concrete piles. The minimum yield strength is 35 ksi.

Corrugated sheet steel (AASHTO M180, Class A, Type 2) is used as rail members for steel railing Type “W”. The minimum yield strength is 50 ksi.

Stainless steel (ASTM A240 Type 304) can be found as sheets on the surface of top plates for Type A and A-T bearings. It is also used for anchor plates cast into the ends of prestressed girders.

The grade of steel, ASTM Specification (or AASHTO Material Specification) associated with the bulleted items listed above (and their components) can be found in the *Bridge Manual*

Chapters or Standards corresponding to these items. This information may also be found in the *Standard Specifications* or "*Special Provisions*".

The modulus of elasticity of steel, E_s , is 29,000 ksi and the coefficient of thermal expansion is 6.5 x 10^{-6} in/in/°F per **LRFD [6.4.1]**.

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11.1 General

11.1.1 Overall Design Process

The overall foundation support design process requires an iterative collaboration to provide cost-effective constructible substructures. Input is required from multiple disciplines including, but not limited to, structural, geotechnical and design. For a typical bridge design, the following four steps are required (see 6.2):

1. Structure Survey Report (SSR) – This design step results in a very preliminary evaluation of the structure type and approximate location of substructure units, including a preliminary layout plan.

2. Site Investigation Report – Based on the Structure Survey Report, a Geotechnical Investigation (see Chapter 10 – Geotechnical Investigation) is required, including test borings to determine foundation requirements. A hydraulic analysis is also performed at this time, if required, to assess scour potential and maximum scour depth. The Site Investigation Report and Subsurface Exploration Drawing are used to identify known constraints that would affect the foundations in regard to type, location or size and includes foundation recommendations to support detailed structural design. Certain structure sites/types may require the preliminary structure plans (Step 3) prior to initiating the geotechnical site investigation. One example of this is a multi-span structure over water. See 6.2 for more information.

3. Preliminary Structure Plans – This design step involves preparation of a general plan, elevation, span arrangement, typical section and cost estimate for the new bridge structure. The Site Investigation Report is used to identify possible poor foundation conditions and may require modification of the structure geometry and span arrangement. This step may require additional geotechnical input, especially if substructure locations must be changed.

4. Final Contract Plans for Structures – This design step culminates in final plans, details, special provisions and cost estimates for construction. The Subsurface Exploration sheet(s) are part of the Final Contract Plans. Unless design changes are required at this step, additional geotechnical input is not typically required to prepare foundation details for the Final Contract Plans.

11.1.2 Foundation Type Selection

The following items need to be assessed to select site-specific foundation types:

- Magnitude and direction of loads.

- Depth to suitable bearing material.

- Potential for liquefaction, undermining or scour.

- Frost potential.

For piles subject to large lateral loads, the structural pile capacity must also be checked for shear and combined stress against flexure and compression.

Piles subject to uplift must also be checked for tension resistance.

A concrete compressive strength of 4 ksi is the minimum value required by specification, while a value of 3.5 ksi is used in the structural design computations. Pile capacities are maximums, based on an assumed concrete compressive strength of 3.5 ksi. The concrete compressive strength of 3.5 ksi is based on construction difficulties and unknowns of placement. The Geotechnical Site Investigation Report must be used as a guide in determining the nominal geotechnical resistance for the pile.

Any structural strength contribution associated with the steel shell is neglected in driven CIP concrete pile design. Therefore, environmentally corrosive sites do not affect driven CIP concrete pile designs. An exception is that CIP should not be used for exposed pile bents in corrosive environments as shown in the *Facilities Development Manual,* Procedure 13-1-15.

Based on the above equation, current WisDOT practice is to design driven cast-in-place concrete piles for factored (ultimate structural) axial compression resistances as shown in Table 11.3-5. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.** The minimum shell thickness is 0.219 inches for straight steel tube and 0.1793 inches for fluted steel shells, unless otherwise noted in the Geotechnical Site Investigation Report and stated in the project plans. Exposed piling (e.g. open pile bents) should not be less than 12 inches in diameter.

When cobbles or other difficult driving conditions are present, the minimum wall thickness for steel shells of driven cast-in-place concrete piles should be increased to 0.25 inches or thicker to facilitate driving without damaging the pile. A drivability analysis should be completed in design, to determine the required wall thickness based on site conditions and an assumed driving equipment.

Driven cast-in-place concrete pile is generally the most favorable displacement pile type since inspection of the steel shell is possible prior to concrete placement and more reliable control of concrete placement is attainable.

11.3.1.12.2.2 Precast Concrete Piles

Precast concrete pile can be divided into two primary types – reinforced concrete piles and prestressed concrete piles. These piles have parallel or tapered sides and are usually of rectangular or round cross section. Since the piles are usually cast in a horizontal position, the round cross section is not common because of the difficulty involved in filling a horizontal cylindrical form. Because of the somewhat variable subsurface conditions in Wisconsin and the need for variable length piles, these piles are currently not used in Wisconsin.

11.3.1.12.3 Steel Piles

Steel pile generally consist of either H-pile or pipe pile types. Both open-end and closed-end pipe pile are used. Pipe piles may be left open or filled with concrete, and can also have a structural shape or reinforcement steel inserted into the concrete. Open-end pipe pile can be socketed into bedrock with preboring.

Steel pile is typically top driven at the pile butt. However, closed-end pipe pile can also be bottom driven with a mandrel. Mandrels are generally not used in Wisconsin.

Steel pile can be used in friction, point-bearing, a combination of both, or rock-socketed piles. One advantage of steel pile is the ease of splicing or cutting to accommodate differing final constructed lengths.

Steel pile should not be used for exposed pile bents in corrosive environments as show in the *Facilities Development Manual, Procedure 13.1.15*.

The nominal (ultimate) axial structural compressive resistance of steel piles is designed in accordance with **LRFD [10.7.3.13.1]** as either non-composite or composite sections. Composite sections include concrete-filled pipe pile and steel pile that is encased in concrete. The nominal structural compressive resistance for non-composite and composite steel pile is further specified in **LRFD [6.9.4 and 6.9.5]**, respectively. The effective length of horizontally unsupported steel pile is determined in accordance with **LRFD [10.7.3.13.4]**. Resistance factors for the structural compression limit state are specified in **LRFD [6.5.4.2]**.

WisDOT policy item:

For steel H-piles, 50 ksi shall be used for pile design. For steel pipe piles, 35 ksi shall be used for pile design and drivability analyses. Plans shall note specified yield strength.

11.3.1.12.3.1 H-Piles

Steel piles are generally used for point-bearing piles and typically employ what is known as the HP-section (often called H-piles for brevity). Steel H-piles are rolled sections with wide flanges such that the depth of the section and the width of the flanges are approximately equal. The cross-sectional area and volume displacement are relatively small and as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered “non-displacement” piling.

H-piles are available in many sizes and lengths. Unspliced pile lengths up to 140 feet and spliced pile lengths up to 230 feet have been driven. Typical pile lengths range from 40 to 120 feet. Common H-pile sizes vary between 10 and 14 inches.

The current WisDOT practice is to design driven H-piles for the factored (ultimate structural) axial compression resistance as shown in Table 11.3-5. These values are based on ϕ_c = 0.5 for severe driving conditions **LRFD [6.5.4.2]**. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the**

Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Resistance (Pn) (tons) (2)(3)(6)	(ϕ)	Maximum Factored Resistance (Pr) (tons) (4)	Modified Gates Driving Criteria: Factored Resistance (Pr) (ϕ = 0.50) (tons)	Modified Gates Driving Criteria: Required Driving Resistance (Rn_{dyn}) (tons) (5)	PDA/CAPWAP Driving Criteria: Factored Resistance (Pr) (ϕ = 0.65) (tons)	PDA/CAPWAP Driving Criteria: Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles									
10 ¾“	0.219	83.5	99.4	0.75	75	55 (8)	110	72 (8)	110
10 ¾“	0.250	82.5	98.2	0.75	74	65 (8)	130	75 (9)	115
10 ¾“	0.365	78.9	93.8	0.75	70	75 (9)	150	75 (9)	115
10 ¾“	0.500	74.7	88.8	0.75	67	75 (9)	150	75 (9)	115
12 ¾”	0.250	118.0	140.4	0.75	105	80 (8)	160	104 (8)	160
12 ¾”	0.375	113.1	134.6	0.75	101	105 (9)	210	104 (9)	160
12 ¾”	0.500	108.4	129.0	0.75	97	105 (9)	210	104 (9)	160
14”	0.250	143.1	170.3	0.75	128	85 (8)	170	111 (8)	170
14”	0.375	137.9	164.1	0.75	123	120 (8)	240	120	185
14”	0.500	132.7	158.0	0.75	118	120 (9)	240	120 (9)	185
16”	0.375	182.6	217.3	0.75	163	145 (8)	290	159	245
16”	0.500	176.7	210.3	0.75	158	160 (9)	320	159 (9)	245
H-Piles									
10 x 42	NA(1)	12.4	310.0	0.50	155	90	180 (10)	117	180 (10)
12 x 53	NA(1)	15.5	387.5	0.50	194	110	220 (10)	143	220 (10)
14 x 73	NA(1)	21.4	535.0	0.50	268	125	250 (10)	162	250 (10)

Table 11.3-5
Typical Pile Axial Compression Resistance Values

Notes:

1. NA – not applicable
2. For CIP Piles: Pn = 0.8 (k_C * f’c * Ag + fy * As) **LRFD [5.7.4.4-3]**. k_C = 0.85 (for $f'_C \leq$ 10.0 ksi). Neglecting the steel shell, equation reduces to 0.68 * f’c * Ag.

   f’c = compressive strength of concrete = 3,500 psi

3. For H-Piles: Pn = (0.66^{λ} * Fe * As) **LRFD [6.9.5.1-1]** (λ = 0 for piles embedded in the ground below the substructure, i.e. no unsupported lengths)

   Fe = fy = yield strength of steel = 50,000 psi

4. $Pr = \phi * Pn$

   ϕ = 0.75 (**LRFD [5.5.4.2.1]** for axial compression concrete)

   ϕ = 0.50 (**LRFD [6.5.4.2]** for axial steel, for difficult driving conditions)

5. The Required Driving Resistance is the lesser of the following:

   - $Rn_{dyn} = Pr / \varphi_{dyn}$

     φ_{dyn} = 0.50 for construction driving criteria using modified Gates dynamic formula

     φ_{dyn} = 0.65 for construction driving criteria using PDA/CAPWAP

   - The maximum allowable driving stress based on 90 percent of the specified yield stress = 35,000 psi for CIP piles and 50,000 psi for H-Piles (see note 10).

6. Values for Axial Compression Resistance are calculated assuming the pile is fully supported. Piling not in the ground acts as an unbraced column. Calculations verify that the pile values given in Table 11.3-5 are valid for open pile bents within the limitations described in 13.2.2. Cases of excessive scour require the piling to be analyzed as unbraced columns above the point of streambed fixity.

7. If less than the maximum axial resistance, P_r, is required by design, state only the required corresponding driving resistance on the plans.

8. The Factored Axial Compression Resistance is controlled by the maximum allowable driving resistance based on 90 percent of the specified yield stress of steel rather than concrete capacity.

9. Values were rounded up to the value above so as to not penalize the capacity of the thicker walled pile of the same diameter. (Wisconsin is conservative in not considering the pile shell in the calculation of the Factored Axial Compression Resistance. Rounded values utilize some pile shell capacity)

10. Rn_{dyn} values given for H-Piles are representative of past Departmental experience (rather than Pn x ø) and are used to avoid problems associated with overstressing during driving. These Rn_{dyn} values result in driving stresses much less than 90 percent (46%-58%) of the specified yield stress. If other H-Piles are utilized that are not shown in the table, driving stresses should be held to approximately this same range.

It is impractical to test every pile on a project. Therefore, test results can be applied to other piles or pile groups providing that the following conditions exist:

- The other piles are of the same type, material and size as the test piles.
- Subsoil conditions are comparable to those at the test pile locations.
- Installation methods and equipment used are the same as, or comparable to, those used for the test piles.
- Piles are driven to the same penetration depth or resistance or both as the test piles to compensate for variations in the vertical position and density of the bearing strata.

11.3.1.19 Construction Monitoring for Economic Evaluation of Deep Foundations

The goal of the foundation design is to provide the most efficient and economical design for the subsurface conditions. The design of pile-supported foundations is influenced by the resistance factor, which is generally a function of pile resistance determination during installation. The discussion in 11.3.1.14 presents the definition of resistance factors. From a practical point of view the resistance factor for a deep foundation is the relationship between the Factored Axial Compression Resistance (FACR) and the Required Driving Resistance (RDR). The potential resistance factors (see Table 11.3-1) for use in deep foundation design are as follows:

Methods Used to Determine Required Driving Resistance	Resistance Factor
FHWA-modified Gates dynamic pile driving formula (end of drive condition only).	0.50
Driving criteria established by dynamic test [Pile Driving Analyzer, (PDA)] with signal matching [CAse Pile Wave Analysis Program, (CAPWAP)] at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles LRFD[Table 10.5.5.2.3-1]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.65
Static Pile Load Test(s) and dynamic test (PDA) with signal matching (CAPWAP) at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles LRFD[Table 10.5.5.2.3-1]. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.80

Table 11.3-6
Resistance Factors and Deep Foundation Methods of Construction Monitoring

The typical method for a majority of the Department's deep foundation substructures is using the modified Gates dynamic formula to determine the RDR and to use a resistance factor of 0.50. A comparison should be made between the use of the modified Gates and the use of the PDA with CAPWAP or the use of the Static Pile Load Test and the PDA with CAPWAP to determine which method is the most economical.

There are two possible methods available to economically use the PDA with CAPWAP to determine the required driving resistance, which allows the use of a resistance factor of 0.65.

> Method 1: Reduce the number of piles in the substructure by driving the piles to the same RDR as using the modified Gates, but then increasing the FACR used in design. This is possible because the department has set a maximum value on the RDR, which when converted to the FACR is less than the structural capacity of the piles. This is true for all H-piles, and for some CIP piles when the FACR is controlled by the maximum allowable compression stress during driving based on 90 percent of the specified yield stress of steel.

> Method 2: Drive each pile to a lower RDR, which should result in a shorter pile length. The number of piles per substructure would remain the same. The design estimated pile lengths are a function of the assumed soil conditions and the required driving resistance. The as-built pile lengths are a function of the actual soil conditions encountered and the contractor's hammer selection.

The department recommends Method 1 when evaluating the potential economic benefits of using the PDA with CAPWAP, because of the difficultly in accurately predicting pile lengths.

The method used to compare modified Gates to Static Pile Load Test(s) and the PDA with CAPWAP, which allows the use of a resistance factor of 0.80, would follow the procedures described in Method 1 used in the PDA with CAPWAP, reducing the number of piles per substructure. The number of static load test(s) will be a function of the size and number of substructures, the general spatial extent of the area in question and the variability of the subsurface conditions in the area of interest.

The costs to be included in the economic evaluation include the cost of the piling, the cost for the Department/Consultant to monitor the test piles, the cost for the Consultant CAPWAP evaluation (the Department does not currently have this capability), the unit costs for the contractor's time for driving and redriving the test piles, and the cost for the static pile load test(s).

Once the investigation of the subsurface conditions has been completed the geotechnical engineer and the structure engineer should discuss the potential for cost savings by increasing the resistance factor. The Bureau of Structures, Geotechnical Engineering Unit and the Region should be included in the discussion and should be part of the decision. Generally, the larger the project, the greater the potential for significant savings. The Department has two PDA's; therefore, the project team should contact the Geotechnical Engineering Unit (608-246-7940) to evaluate resources prior to incorporation of an increased resistance factor in the foundation design. PDA monitoring may be completed by Department or consultant personnel.

The following two examples use Method 1 to illustrate the potential cost savings/expenses for PDA with CAPWAP:

Pier
Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot. (**Note:** It is realized that for pier design the number of piles is not exclusively related to the vertical load, but this example is simplified for illustrative purposes).
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 110 tons = 32 piles <u>Pile Cost = 32 piles x 100 feet x $40/ft = $128,000</u> Total Cost = $128,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles Pile Cost = 25 piles x 100 feet x $40/ft = $100,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 <u>CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400</u> Total Cost = $103,000
PDA/CAPWAP Savings = $25,000/pier
Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Abut = 980 tons, Number of Piles = 980 tons / 110 tons = 9 piles Total Cost = 9 piles x 100 feet x $40/ft = $36,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 143 tons = 7 piles, however because of maximum spacing requirements the design will

need 8 piles.

Pile Cost = 8 piles x 100 feet x $40/ft = $32,000
PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400
PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200
CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400
Total Cost = $35,000

PDA/CAPWAP Cost = $1000/abutment

Note: For a three span bridge, with 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $52,000. For a two span bridge, with 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $5,400. Bid prices based on 2014-2015 cost data.

Table 11.3-7
Economical Evaluation for Deep Foundations with Two Construction Monitoring Methods

11.3.2 Drilled Shafts

11.3.2.1 General

Drilled shafts are generally large diameter, cast-in-place, open ended, cased concrete piles which are designed to carry extremely heavy loads. Drilled shafts can be the most economical foundation alternative at sites where foundation loads are carried to bearing on dense strata or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or expensive to construct, and where high overturning moments must be resisted.

Drilled shafts are installed by removing soil and rock using drilling methods or other excavation techniques and constructing the foundation element in the excavated hole. The excavated hole may be supported using temporary or permanent casing, drilling slurry or other methods. The hole is then filled with a reinforcement cage and cast-in-place concrete. Drilled shafts are non-displacement elements since the soil volume required for the element is physically removed prior to installation. Thus the effective normal stress adjacent to the pile remains unchanged or is reduced (due to expansion of the soil into the hole before insertion/construction of the load bearing element), and the soil properties and pore water pressure adjacent to the foundation elements are not significantly impacted.

Because drilled shafts do not require a hammer for installation and do not displace the soil, they typically have much less impact on adjacent structures. Depending on the excavation technique used, they can penetrate significant obstructions. Because the method of construction often allows a decrease in the effective stress immediately adjacent to and beneath the tip of the foundation element, the resistance developed will often be less than an equivalently sized driven pile.

11.3.3 Micropiles

11.3.3.1 General

In areas of restricted access, close proximity to settlement sensitive existing structures or difficult geology, micropiles may be considered when determining the recommended foundation type. Although typically more expensive than driven pile, constructability considerations may warrant selection of micropiles as the preferred foundation type. A micropile is constructed by drilling a borehole with drill casing, placing reinforcement and grouting the hole. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can be installed in areas with restricted access and vertical clearance. Drill casing permits installation in poor ground conditions. Micropiles are installed with the same type of equipment that is used for ground anchor and grouting projects. Micropiles can be either vertical or battered.

Micropiles are used for structural support of new structures, underpinning existing structures, scour protection and seismic retrofit at existing structures. Micropiles are also used to create a reinforced soil mass for ground stabilization.

With a micropile's smaller cross-sectional area, the pile design is more frequently governed by structural and stiffness considerations. Due to the small pile diameter, point resistance is usually disregarded for design. Steel casing for micropiles is commonly delivered in 5 to 20 foot long flush-joint threaded sections. The casing is typically 5.5 to 12 inches in diameter, with yield strength of 80 ksi. Grout is mixed neat with a water/cement ratio on the order of 0.45 and an unconfined compressive strength of 4 to 6 ksi. Grade 60, 90 and 150 single reinforcement bars are generally used with centralizers.

Grout/ground bond capacity varies directly with the method of placement and pressure used to place the grout. Common methods include grout placement under gravity head, grout placement under low pressure as temporary drill steel is removed and grout placement under high pressure using a packer and regrout tube. Some regrout tubes are equipped to allow regrouting multiple times to increase pile capacity.

11.3.3.2 Design Guidance

Micropiles shall be designed in conformance with the current *AASHTO LRFD* and in accordance with the WisDOT Bridge Manual. Design guidelines for micropiles are provided in FHWA Publication No. FHWA-NHI-05-039.

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

Augered cast-in-place (ACIP) piles are installed by drilling a hole with a hollow stem auger. When the auger reaches a design depth (elevation) or given torque, sand-cement grout or concrete is pumped through the hollow-stem auger while the auger is withdrawn from the ground. Reinforcement steel can be placed while the grout is still fluid. A single reinforcement bar can also be installed inside the hollow stem auger before the auger is extracted. ACIP piles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can also be installed in areas with restricted access and vertical clearance. Temporary casing is not required. In many situations, these foundation systems can be constructed more quickly and less expensively than other deep foundation alternatives.

ACIP piles are generally available in 12- to 36-inch diameters and typically extend to depths of 60 to 70 feet. In some cases, ACIP piles have been installed to depths of more than 100 feet. The torque capacity of the drilling equipment may limit the available penetration depth of ACIP piles, especially in stiff to hard cohesive soil. Typical Wisconsin bridge contractors do not own the necessary equipment to install this type of pile.

ACIP piles may be more economical; however, there is a greater inherent risk in their installation from the quality control standpoint. There is currently no method available to determine pile capacity during construction of ACIP piles. WisDOT does not generally use this pile type unless there are very unusual design/site requirements.

11.3.4.2 Design Guidance

In the future, the FHWA will distribute a Geotechnical Engineering Circular that will provide design and construction guidance for ACIP piles. WisDOT plans to reassess the use of ACIP piles at that time.

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Pile bents may only be specified where the structure is located within Area 3, as shown in the *Facilities Development Manual 13-1-15*, *Attachment 15.1* and where the piles are not exposed to water with characteristics that are likely to cause accelerated corrosion.

The minimum cap size shall be 3' wide by 3'-6" deep and the piles shall be embedded into the cap a minimum of 2'-0.

13.2.3 Pile Encased Piers

Pile encased piers are similar to pile bents except that a concrete encasement wall surrounds the piles. They are most commonly used for small to intermediate stream crossings where a pile bent pier is not feasible. Pile encased piers are detailed on Standard for Pile Encased Pier.

An advantage of this pier type is that the concrete encasement wall provides greater resistance to lateral forces than a pile bent. Also the hydraulic characteristics of this pier type are superior to pile bents, resulting in a smoother flow and reducing the susceptibility of the pier to scour at high water velocities. Another advantage is that floating debris and ice are less likely to accumulate against a pile encased pier than between the piles of a pile bent. Debris and ice accumulation are detrimental because of the increased stream force they induce. In addition, debris and ice accumulation cause turbulence at the pile, which can have the effect of increasing the local scour potential.

Pile sections shall be limited to 10", 12" or 14" steel HP piles, or 10¾", 12¾" or 14" diameter cast-in-place concrete piles with steel shells. Minimum center-to-center spacing is 3'. Where difficult driving conditions are expected, oil field pipe may be specified in the design. A minimum of five piles per pier shall be used. When a satisfactory design cannot be developed with one of these pile sections at the minimum spacing, another pier type should be selected. The inside and outside piles shall be driven vertically.

In most cases, a cofferdam should be used for pile encased piers. See 13.11.5 for additional guidance regarding cofferdams. Total pier height shall be less than 25 feet.

All bearings supporting a superstructure utilizing pile encased piers shall be fixed bearings or semi-expansion.

The connection between the superstructure and the pier shall be designed to transmit the portion of the superstructure design loads assumed to be taken by the pier.

The concrete wall shall be a minimum of 2'-6" thick. The top 3' of the wall is made wider if a larger bearing area is required. See Standard for Pile Encased Pier for details. The bottom of the wall shall be placed 2' to 4' below stable streambed elevation, depending upon stream velocities and frost depth.

13.2.4 Solid Single Shaft / Hammerheads

Solid single shaft piers are used for all types of crossings and are detailed on Standards for Hammerhead Pier and for Hammerhead Pier – Type 2. The choice between using a multi-column pier and a solid single shaft pier is based on economics and aesthetics. For high level bridges, a solid single shaft pier is generally the most economical and attractive pier type available.

The massiveness of this pier type provides a large lateral load capacity to resist the somewhat unpredictable forces from floating ice, debris and expanding ice. They are suitable for use on major rivers adjacent to shipping channels without additional pier protection. When used adjacent to railroad tracks, crash walls are not required.

If a cofferdam is required and the upper portion of a single shaft pier extends over the cofferdam, an optional construction joint is provided 2' above the normal water elevation. Since the cofferdam sheet piling is removed by extracting vertically, any overhead obstruction prevents removal and this optional construction joint allows the contractor to remove sheet piling before proceeding with construction of the overhanging portions of the pier.

A hammerhead pier shall not be used when the junction between the cap and the shaft would be less than the cap depth above normal water. Hammerhead piers are not considered aesthetically pleasing when the shaft exposure above water is not significant. A feasible alternative in this situation would be a wall type solid single shaft pier or a multi-column pier. On a wall type pier, both the sides and ends may be sloped if desired, and either a round, square or angled end treatment is acceptable. If placed in a waterway, a square end type is less desirable than a round or angled end.

13.2.5 Aesthetics

Refer to 13.13 for suggested alternative pier shapes. These shapes are currently being studied so no standard details are shown. It is desirable to standardize alternate shapes for efficiency and economy of construction. Use of these alternate pier shapes for aesthetics should be approved by the Chief Structures Development Engineer so that standard details can be developed.

Refer to Chapter 4 for additional information about aesthetics.

To prevent unequal settlement, proportion the continuous footing so that soil pressures or pile loads are constant for Service I load combination. The footing should be kept relatively stiff between columns to prevent upward footing deflections which cause excessive soil or pile loads under the columns.

13.11.5 Cofferdams and Seals

A cofferdam is a temporary structure used to construct concrete substructures in or near water. The cofferdam protects the substructure during construction, controls sediments, and can be dewatered to construct the substructure in a dry environment. Dewatering the cofferdam allows for the cutting of piles, placement of reinforcing steel and ensuring proper consolidation of concrete. A cofferdam typically consists of driven steel sheet piling and allows for the structure to be safely dewatered when properly designed. Alternative cofferdam systems may be used to control shallow water conditions.

A cofferdam bid item may be warranted when water is expected at a concrete substructure unit during construction. The cofferdam shall be practically watertight to allow for dewatering such that the substructure is constructed in a dry environment. An exception is for pile encased piers with expected water depths of 5 feet or less. These substructures may be poured underwater, but in certain cases may still require a cofferdam for protection and/or to address environmental concerns. A pile encased pier with expected water depths greater than 5 feet will typically require a cofferdam. The designer should consult with geotechnical and regional personnel to determine if a cofferdam is required. If a cofferdam is warranted, then include the bid item “Cofferdams (Structure)”.

Environmental concerns (specifically sediment control) may require the use of cofferdams at some sites. When excavation takes place in the water, some form of sediment control is usually required. The use of simple turbidity barrier may not be adequate based on several considerations (water depth, velocity, soil conditions, channel width, etc.). All sediment control devices, such as turbidity barrier, shall not be included in structure plans. Refer to Chapter 10 of the FDM for erosion control and storm water quality information.

A seal is a mat of unreinforced concrete poured under water inside a cofferdam. The seal is designed to withstand the hydrostatic pressure on its bottom when the water above it is removed. For shallow water depths and certain soil conditions a concrete seal may not be necessary in order to dewater a cofferdam. Coordinate with geotechnical personnel to determine if a concrete seal is required. The designer shall determine if a concrete seal is required for a cofferdam. If a concrete seal is required, then include the bid item “Concrete Masonry Seal” and required seal dimensions. The cofferdam design shall be the responsibility of the contractor.

The hydrostatic pressure under the seal is resisted by the seal weight, the friction between the seal perimeter and the cofferdam walls, and friction between seal and piles for pile footings. The friction values used for the seal design are considered using the service limit state. To compute the capacity of piles in uplift, refer to Chapter 11. Values for bond on piles and sheet piling are presented in Table 13.11-1.

Application	Value of Bond
Bond on Piles	10 psi
Bond on Sheet Piling	2 psi applied to [(Seal Depth - 2') x Seal Perimeter]

Table 13.11-1
Bond on Piles and Sheet Piling

Lateral forces from stream flow pressure are resisted by the penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. When seals for spread footings are founded on rock, the weight of the seal is used to counterbalance the lateral stream flow pressure.

The downstream side of the cofferdam should be keyed into rock deep enough or other measures should be used to resist the lateral stream flow pressure. To provide a factor of safety, the cofferdam weight (sheet piling and bracing) is ignored in the analysis. The design stream flow velocity is based on the flow at the site at the time of construction but need not exceed 75% of the 100-year velocity. The force is calculated as per 13.4.6.

A rule of thumb for seal thickness is 0.40H for spread footings and 0.25H for pile footings, where H is the water depth from bottom of seal to top of water. The 2-year high water elevation, if available, should be used as the estimated water elevation during construction. <u>The assumed water elevation used to determine the seal thickness should be noted on the plans</u>. The minimum seal size is 3'-0" larger than the footing size on all sides. See Standard for Hammerhead Pier for additional guidance regarding the sizing of the seal.

Example: Determine the seal thickness for a 9’ x 12’ footing with 12-12” diameter piles. Uplift capacity of one pile equals 10 kips. The water depth to the top of seal is 16’.

Assume 12’ x 15’ x 3’ seal.

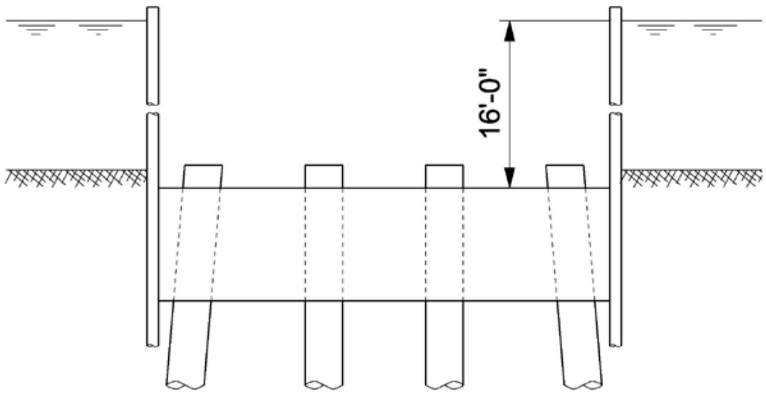

Figure 13.11-4
Seal Inside a Cofferdam

Uplift force of water	12 x 15 x 19 x .0624	=	214 kips up
Weight of seal course	12 x 15 x 3 x .15	=	81 kips down
Friction of sheet piling	2 x (12+15) x 1 x 144 x .002	=	2 x (12+15) x 1 x 144 x .002
		=	16 kips down
Friction on 12-inch diameter pile	p x 12 x 36 x .010	=	13.6 kips
Maximum uplift/pile		=	10 kips
Total available force from piles	12 x 10	=	120 kips down
Summation of downward forces		=	217 kips
	217 > 214 OK		

USE 3’-0” THICK SEAL

13.12 Quantities

Consider the "Upper Limits for Excavation" for piers at such a time when the quantity is a minimum. This is either at the existing ground line or the finished graded section. Indicate in the general notes which value is used.

Granular backfill is not used at piers except under special conditions.

Compute the concrete quantities for the footings, columns and cap, and show values for each of them on the final plans.

13.13 Design Examples

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The non-metallic or extensible reinforcement includes the following:

Geogrids: The geogrids are mostly used with modular block walls.

Geotextile Reinforcement: High strength geotextile can be used principally with wrap-around and temporary wall construction.

Corrosion of the wall anchors that connect the soil reinforcement to the wall face must also be accounted for in the design.

14.6.2.3 Facing Elements

The types of facings element used in the different MSE walls mainly control aesthetics, provide protection against backfill sloughing and erosion, and may provide a drainage path in certain cases. A combination of different wall facings and reinforcement provide a choice of selecting an MSE wall which can be used for several different functions.

Major facing types are:

- Segmental precast concrete panels
- Dry cast or wet cast modular blocks
- Full height pre-cast concrete panels (tilt-up)
- Cast-in-place concrete facing
- Geotextile-reinforced wrapped face
- Geosynthetic /Geogrid facings
- Welded wire grids

Segmental Precast Concrete Panels

Segmental precast concrete panels include small panels (<30 sq ft) to larger (>30 sq ft) with a minimum thickness of 5-½ inches and are of a square, rectangular, cruciform, diamond, or hexagonal geometry. The geometric pattern of the joints and the smooth uniform surface finish of the factory provided precast panels give an aesthetically pleasing appearance. Segmental precast concrete panels are proprietary wall components.

Wall panels are available in a plain concrete finish or numerous form liner finishes and textures. An exposed aggregate finish is also available along with earth tone colors. Although color can be obtained by adding additives to the concrete mix it is more desirable to obtain color by applying concrete stain and/or paint at the job site. Aesthetics do affect wall costs.

WisDOT requires that MSE walls utilize precast concrete panels when supporting traffic live loads which are in close proximity to the wall. Panels are also allowed as components of an abutment structure. Either steel strips or welded wire fabric is allowed for soil reinforcement when precast concrete panels are used as facing of the MSE wall system.

Walls with curved alignments shall limit radii to 50 feet for 5 feet wide panels and 100 feet for 10 feet wide panels. Typical joint openings are not suitable for wall alignments following a tighter curve. Special joints or special panels that are less than 5 feet wide may be able to accommodate tighter curves. In general, MSE wall structures with panel type facings shall be limited to wall heights of 33 feet. Contact Structures Design Section for approval on case by case basis.

Concrete Modular Blocks Facings

Concrete modular block retaining walls are constructed from modular blocks typically weighing from 40 to 100 pounds each, although blocks over 200 pounds are rarely used. Nominal front to back width ranges between 8 to 24 inches. Modular blocks are available in a large variety of facial textures and colors providing a variety of aesthetic appearances. The shape of the blocks usually allows the walls to be built along a curve, either concave (inside radius) or convex (outside radius). The blocks or units are dry stacked meaning mortar or grout is not used to bond the units together except for the top two layers. Figure 14.6-2 shows various types of blocks available commercially. Figure 14.6-3 shows a typical modular block MSE wall system along with other wall components. Most modular block MSE walls are reinforced with geogrids.

Modular blocks can be either dry cast or wet cast. Dry cast (small) blocks are mass produced by using a zero slump concrete that allows forms to be stripped faster than wet cast (large) blocks. MSE walls usually use dry cast blocks since they are usually a cheaper facing and wall stability is provided by the reinforced mass. Gravity walls rely on facing size and mass for wall stability. For minor walls dry cast blocks are typically used and for taller gravity walls wider wet cast blocks are normally required to satisfy stability requirements.

Concrete modular blocks are proprietary wall component systems. Each proprietary system has its own unique method of locking the units together to resist the horizontal shear forces that develop. Fiberglass pins, stainless steel pins, glass filled nylon clips and mechanical interlocking surfaces are some of the methods utilized. Any pins or hardware must be manufactured from corrosion resistant materials.

During construction of these systems, the voids are filled with granular material such as crushed stone or gravel. Most of the systems have a built in or automatic set-back (incline angle of face to the vertical) which is different for each proprietary system. Blocks used on WisDOT projects must be of one piece construction. A minimum weight per block or depth of block (distance measured perpendicular to wall face) is not specified on WisDOT projects. The minimum thickness allowed of the front face is 4 inches (measured perpendicular from the front face to inside voids greater than 4 square inches). Also the minimum allowed thickness of any other portions of the block (interior walls or exterior tabs, etc.) is 2 inches.

Alignments that are not straight (i.e. kinked or curved) shall use 90 degree corners or curves. The minimum radius should be limited to 8 feet. For a concave wall the radius is measured to the front face of the bottom course. For convex walls the radius is measured to the front face of the top course. In no case shall the radius be less than 6 feet. It is WisDOT policy to design modular block MSE walls for a maximum height of 22 ft (measured from the top of the leveling pad to the top of the wall).

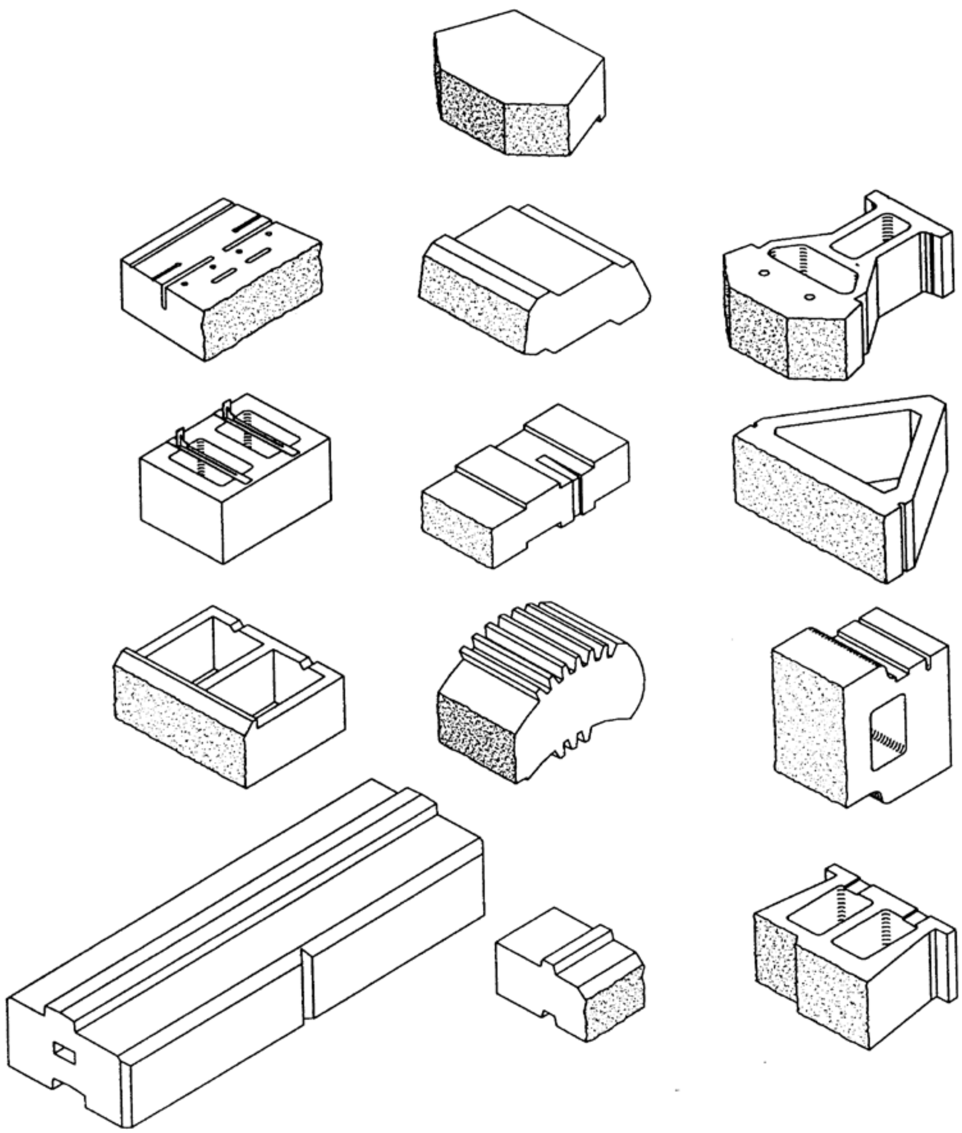
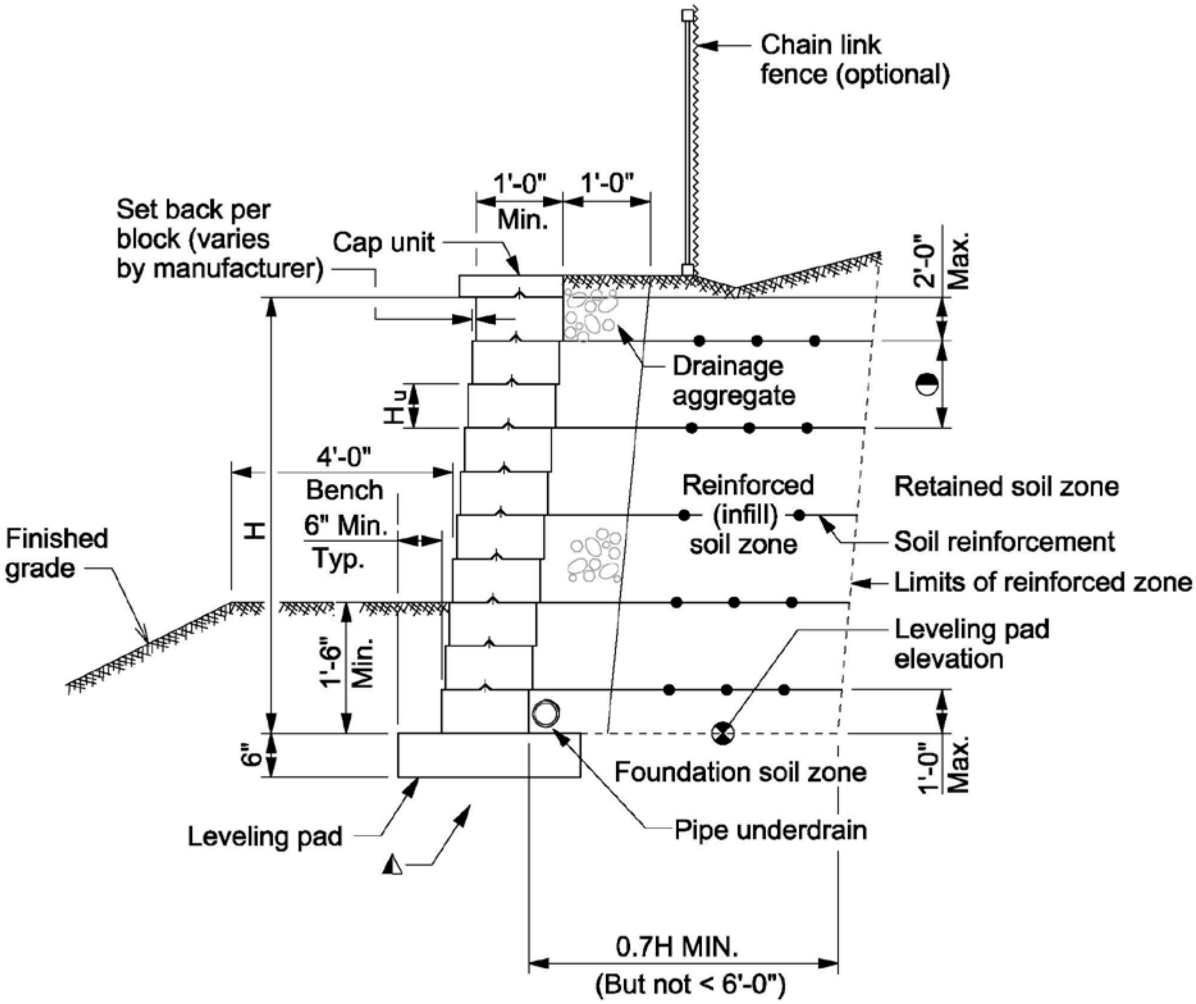


Figure 14.6-2
Modular Blocks
(Source FHWA-NHI-10-025)



Modular Block MSE Wall

▲ Ground improvement meaures should be taken when the soil below the levelling pad is poor or subject to frost heave.

◑ Maximum vertical spacing of soil reinforcement layers shall be two times the block depth (Hu) or 32 inches, whichever is less.

Figure 14.6-3
Typical Modular Block MSE Walls

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- Evaluate alternative designs on the basis of competitive materials appropriate to a specific structure type.

- Do not propose specific construction methods or erection procedures in the plans unless constraints are necessary to meet specific project requirements.

- Make an economic evaluation of preliminary estimates based on state-of-the-art methods of construction for structure types.

- Consider future structure maintenance needs in the structure's design in order to provide life-cycle costing data.

- Consider alternate plans where experience, expertise and knowledge of conditions clearly indicate that they are justified. Alternate plans are not compatible with stage construction and should not be used in these situations.

- Value engineering concepts are recognized as being cost effective. Apply these concepts to the selection of structure type, size and location throughout the plan development process.

17.4 Superstructure Types

Superstructures are classified as deck or through types.

For deck type structures, the roadway is above or on top of the supporting structures. Examples of deck type structures are girder bridges and steel deck-trusses.

For through type structures, the roadway passes between two elements of the superstructure. Examples of through type structures are steel through-trusses and tied-arch bridges.

Through type structures are generally used where long span lengths are required. Deck type structures are more common, because they lend themselves to future widening if increased traffic requires it.

Some of the various types of superstructures used in Wisconsin are as follows:

1. Concrete slab (flat and haunched)

   Concrete slab structures are adaptable to roadways with a high degree of horizontal curvature. This superstructure type is functional for short to medium span lengths and is relatively economical to construct and maintain. The practical range of span lengths for concrete slab structures can be increased by using haunched slab structures.

WisDOT policy item:

Concrete slab structures are limited to sites requiring a skew angle of 30 degrees or less.

   Voided slab structures are not currently being used due to excessive longitudinal cracking over the voids in the negative moment region. For more information about concrete slab structures, refer to Chapter 18 – Concrete Slab Structures.

2. Prestressed concrete girder

   Prestressed concrete girder structures are very competitive from a first cost standpoint and require very little maintenance. Prestressed concrete girders are produced by a fabrication plant certified by WisDOT. Future widening can be accomplished with relative ease. For more information about prestressed concrete girder bridges, refer to Chapter 19 – Prestressed Concrete.

3. Concrete T-beam

WisDOT policy item:

The concrete T-beam has had limited use in Wisconsin during recent years and is no longer used.

4. Prestressed box girder

   Prestressed box girder structures have the advantage of rapid construction where traffic must be diverted. Elimination of the need for falsework is a particular advantage when vertical clearances are critical during the construction phase. Experience indicates that, from a first-cost standpoint, these structures are more expensive to construct than concrete slab structures. For more information, refer to Chapter 19 – Prestressed Concrete.

5. Concrete box girder

   The concrete box girder structure is aesthetically adaptable for urban sites having roadways with a high degree of horizontal curvature or large skew angles. This structure is frequently employed in multi-level interchanges where horizontal clearances are limited, since the pier cap is an integral part of the superstructure. However, problems can be encountered in maintenance with deck replacements requiring shoring.

6. Concrete rigid frame

   The concrete rigid frame is more costly than other superstructure types. However, the concrete rigid frame is known for its aesthetic value and is used primarily in public parks and urban areas where the span lengths are similar to concrete slab structures and where approach embankments are relatively high.

7. Steel rolled section and welded plate girder

   Welded plate girders are less expensive than rolled sections with cover plates because of their reduced allowable design stress resulting from the fatigue criteria. Welded plate girders have greater versatility in allowing variable web thickness and depth, as well as variable flange thicknesses. Future widening can be accomplished with relative ease. For more information, refer to Chapter 24 – Steel Girder Structures and Chapter 38 – Railroad Structures.

8. Steel box girder

   Steel box girder structures have span length capabilities similar to plate girders. Aesthetically, they present a smooth, uncluttered appearance due to their closed box sections. Current experience reveals that steel box girders require more material than conventional steel plate girders. However, due to redundancy concerns, use of single-box and two-box girder bridges should be avoided unless absolutely necessary.

9. Steel tied arch and steel truss

   Unusual bridge sites, such as major river and harbor crossings, may require the use of longer span lengths than conventional deck type superstructures can accommodate. For such conditions, a steel tied arch or a steel truss can be used effectively.

10. Timber longitudinally laminated decks

    Timber structures blend well in natural settings and are relatively easy to construct with light construction equipment. Timber longitudinally laminated deck structures have low profiles that generally provide large clearances for high water. Their application is limited by the range of span lengths and economics in comparison to concrete slabs. For more information, refer to Chapter 23 – Timber Structures.

Therefore:

Additional area of steel required = 0.542 - 0.43 = 0.112 in^2/ft

Use either one or two times the spacing of the standard transverse reinforcement.

Lapping every other bar: use #4's @ 17", As = 0.14 in^2/ft, use Detail “A”.

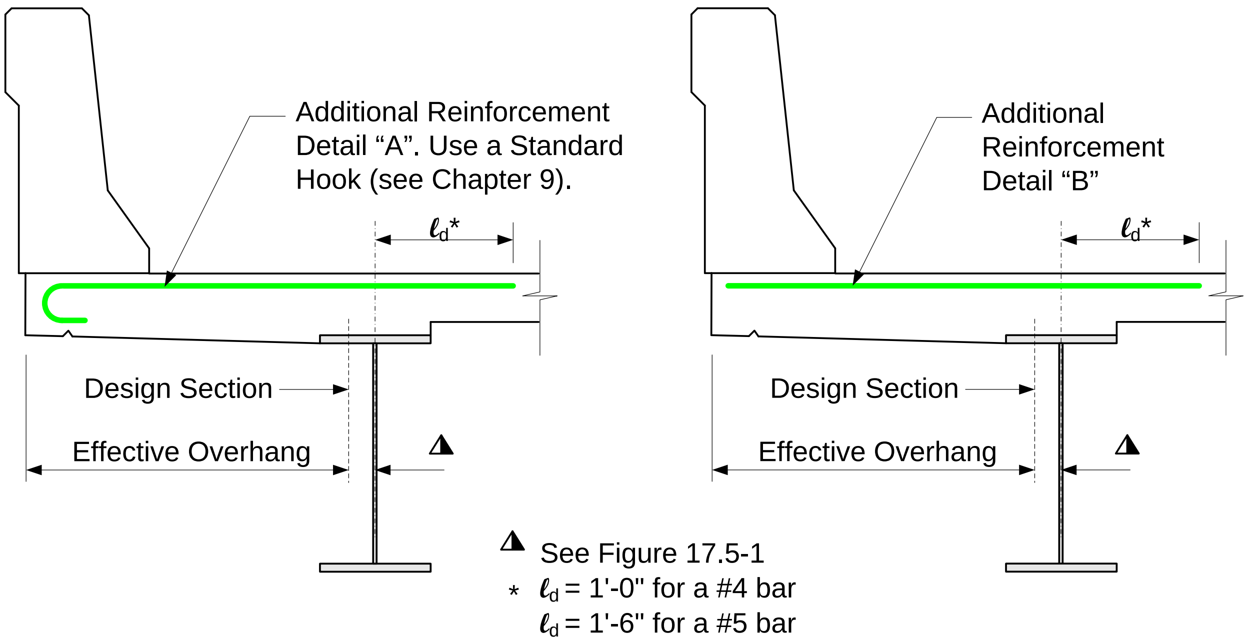

Figure 17.6-8

Overhang Reinforcement Details

To reiterate:

1. Details for additional overhang reinforcement are shown in Figure 17.6-8. Detail “A” shall be used with Table 17.6-2, Table 17.6-3 and Table 17.6-5. Detail “B” shall be used with Table 17.6-4.

2. Girder Type 1 shall include steel girders and the following prestressed girders; 28-inch, 36-inch, 36W-inch, 45W-inch. Prestressed girders used for rehabilitation projects, 45-inch, 54-inch and 70-inch, shall also be considered to be Girder Type 1. Girder Type 2 shall include the following prestressed girders; 54W-inch, 72W-inch and 82W-inch.

17.7 Construction Joints

Optional transverse construction joints are permitted on continuous concrete deck structures to limit the concrete volume in a single pour. Refer to the Standard Detail for Slab Pouring Sequence for the optimum slab pouring sequence. On steel structures over 300 feet long, transverse construction joints, if used, are to be placed at 0.6 of the span length beyond the pier in the direction of the pour. For continuous prestressed concrete girder bridges, optional transverse construction joints should be located midway between the cut-off points for continuity reinforcing steel or at 0.75 of the span, whichever is closest to the pier.

The rate of placing concrete for continuous steel girders shall equal or exceed 0.5 of the span length per hour but need not exceed 100 cubic yards per hour. Transverse construction joints may be omitted with approval of Bureau of Structures.

When the deck width of a girder superstructure exceeds 90 feet or the width of a slab superstructure exceeds 52 feet, a longitudinal construction joint with reinforcement through the joint shall be detailed. Longitudinal joints should not be located directly above girders and should be at least 6 inches from the edge of the top flange of the girder. Longitudinal joints are preferably located beneath the median or parapet. Otherwise, the joint should be located along the edge of the lane line or in the middle of the lane. The longitudinal construction joint should be used for staged construction and for other cold joint applications within the deck. A longitudinal construction joint detail is provided in Standard Detail 17.02 – Deck and Slab Details.

Optional longitudinal construction joints shall be detailed accordingly in the plans. Longitudinal construction joints requested by the contractor are to be approved by the engineer. Optional and contractor requested joints are to be located as previously mentioned.

Open joints may be used in a median or between parapets. Considerations should be given to sealing open joints with compression seals or other sealants.

The structure plans should permit the contractor to propose an alternate construction joint schedule subject to approval of the engineer.

Table of Contents

19.1 Introduction

This chapter provides information intended for prestressed I-girders. Prestressed box girders and general prestressed concrete guidelines are also included in this chapter.

The definition of prestressed concrete as given by the ACI Committee on Prestressed Concrete is:

> "Concrete in which there has been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desired degree. In reinforced concrete members the prestress is commonly introduced by tensioning the steel reinforcement."

This internal stress is induced into the member by either of the following prestressing methods.

19.1.1 Pretensioning

In pretensioning, the tendons are first stressed to a given level and then the concrete is cast around them. The tendons may be composed of wires, bars or strands.

The most common system of pretensioning is the long line system, by which a number of units are produced at once. First the tendons are stretched between anchorage blocks at opposite ends of the long stretching bed. Next the spacers or separators are placed at the desired member intervals, and then the concrete is placed within these intervals. When the concrete has attained a sufficient strength, the steel is released and its stress is transferred to the concrete via bond.

19.1.2 Post-Tensioning

In post-tensioning, the concrete member is first cast with one or more post-tensioning ducts or tubes for future insertion of tendons. Once the concrete is sufficiently strong, the tendons are stressed by jacking against the concrete. When the desired prestress level is reached, the tendons are locked under stress by means of end anchorages or clamps. Subsequently, the duct is filled with grout to protect the steel from corrosion and give the added safeguard of bond.

In contrast to pretensioning, which is usually incorporated in precasting (casting away from final position), post-tensioning lends itself to cast-in-place construction.

19.2 Basic Principles

This section defines the internal stress that results from either prestressing method.

First consider the simple beam shown in Figure 19.2-1.

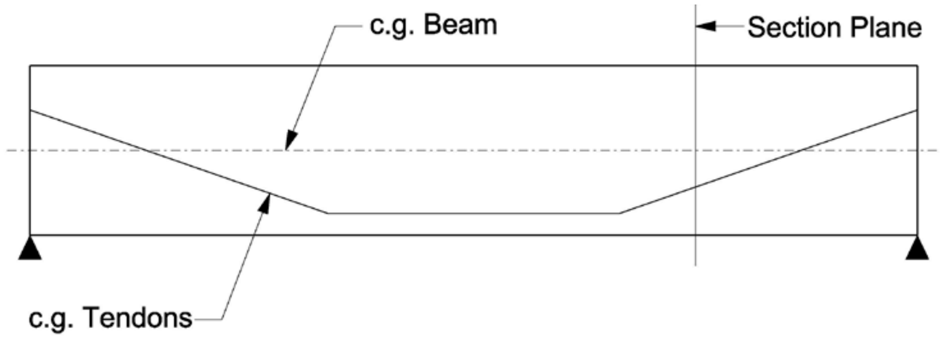

Figure 19.2-1
Simple Span Prestressed Concrete Beam

The horizontal component, P, of the tendon force, F, is assumed constant at any section along the length of the beam.

Also, at any section of the beam the forces in the beam and in the tendon are in equilibrium. Forces and moments may be equated at any section.

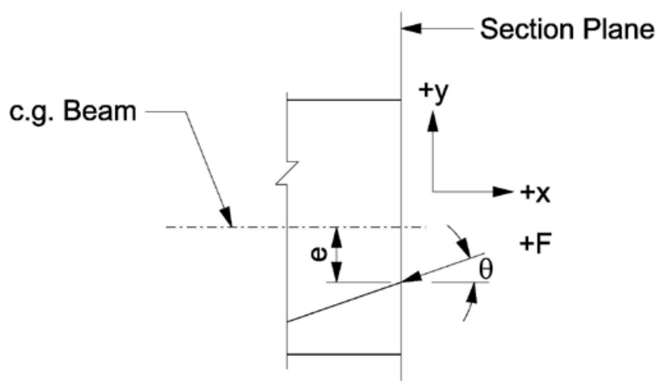

Figure 19.2-2
Assumed Sign Convention for Section Forces

The assumed sign convention is as shown in Figure 19.2-2 with the origin at the intersection of the section plane and the center of gravity (centroidal axis) of the beam. This convention indicates compression as positive and tension as negative.

19.3 Pretensioned Member Design

This section outlines several important considerations associated with the design of conventional pretensioned members.

19.3.1 Design Strengths

The typical specified design strengths for pretensioned members are:

Prestressed I-girder concrete:	f'_c	= 6 to 8 ksi
Prestressed box girder concrete:	f'_c	= 5 ksi
Prestressed concrete (at release):	f'_{ci}	= 0.75 to 0.85 $f'_c \leq 6.8$
Deck and diaphragm concrete:	f'_c	= 4 ksi
Prestressing steel:	f_{pu}	= 270 ksi
Grade 60 reinforcement:	f_y	= 60 ksi

The *actual required* compressive strength of the concrete at prestress transfer, f'_{ci}, is to be stated on the plans. For typical prestressed girders, $f'_{ci(min)}$ is 0.75(f'_c).

WisDOT policy item:

For prestressed I-girders, the use of concrete with strength greater than 8 ksi is only allowed with the prior approval of the BOS Development Section. Occasional use of strengths up to 8.5 ksi may be allowed. Strengths exceeding these values are difficult for local fabricators to consistently achieve as the coarse aggregate strength becomes the controlling factor.

For prestressed box girders, the use of concrete with strength greater than 5 ksi is only allowed with prior approval of the BOS Development Section.

The use of 8 ksi concrete for prestressed I-girders and 6.8 ksi for f'_{ci} still allows the fabricator to use a 24-hour cycle for girder fabrication. There are situations in which higher strength concrete in the prestressed I-girders may be considered for economy, provided that f'_{ci} does not exceed 6.8 ksi. Higher strength concrete may be considered if the extra strength is needed to avoid using a less economical superstructure type or if a shallower girder can be provided and its use justified for sufficient reasons (min. vert. clearance, etc.) Using higher strength concrete to eliminate a girder line is not the preference of the Bureau of Structures. It is often more economical to add an extra girder line than to use debonded strands with the minimum number of girder lines. After the number of girders has been determined, adjustments in girder spacing should be investigated to see if slab thickness can be minimized and balance between interior and exterior girders optimized.

Prestressed I-girders below the required 28-day concrete strength (or 56-day concrete strength for f'_c = 8 ksi) will be accepted if they provide strength greater than required by the design and at the reduction in pay schedule in the *Wisconsin Standard Specifications for Highway and Structure Construction*.

Low relaxation prestressing strands are required.

19.3.2 Loading Stages

The loads that a member is subjected to during its design life and those stages that generally influence the design are discussed in **LRFD [5.9]** and in the following sections. The allowable stresses at different loading stages are defined in **LRFD [5.9.3]** and **LRFD [5.9.4]**.

19.3.2.1 Prestress Transfer

Prestress transfer is the initial condition of prestress that exists immediately following the release of the tendons (transfer of the tendon force to the concrete). The eccentricity of the prestress force produces an upward camber. In addition, a stress due to the dead load of the member itself is also induced. This is a stage of temporary stress that includes a reduction in prestress due to elastic shortening of the member.

19.3.2.2 Losses

After elastic shortening losses, the external loading is the same as at prestress transfer. However, the internal stress due to the prestressing force is further reduced by losses resulting from relaxation due to creep of the prestressing steel together with creep and shrinkage of the concrete. It is assumed that all losses occur prior to application of service loading.

LRFD [5.9.5] provides guidance about prestress losses for both pretensioned and post-tensioned members. This section presents a refined and approximate method for the calculation of time-dependent prestress losses such as concrete creep and shrinkage and prestressing steel relaxation.

WisDOT policy item:

WisDOT policy is to use the approximate method described in **LRFD [5.9.5.3]** to determine time-dependent losses, since this method does not require the designer to assume the age of the concrete at the different loading stages.

Losses for pretensioned members that are considered during design are listed in the following sections.

19.3.2.2.1 Elastic Shortening

Per **LRFD [5.9.5.2.3a]**, the loss due to elastic shortening, Δf_{pES1} (ksi), in pretensioned concrete members shall be taken as:

$$\Delta f_{pES1} = \frac{E_p}{E_{ct}} f_{cgp}$$

Where:

E_p	=	Modulus of elasticity of prestressing steel = 28,500 ksi **LRFD [5.4.4.2]**
E_{ct}	=	Modulus of elasticity of concrete at transfer or time of load application in ksi (see 19.3.3.8)
f_{gcp}	=	Concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)

19.3.2.2.2 Time-Dependent Losses

Per **LRFD [5.9.5.3],** an estimate of the long-term losses due to steel relaxation as well as concrete creep and shrinkage on standard precast, pretensioned members shall be taken as:

$$\Delta f_{pLT} = 10.0 \frac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR}$$

Where:

$$\gamma_h = 1.7 - 0.01H$$

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})}$$

f_{pi}	=	Prestressing steel stress immediately prior to transfer (ksi)
H	=	Average annual ambient relative humidity in %, taken as 72% in Wisconsin
Δf_{pR}	=	Relaxation loss estimate taken as 2.4 ksi for low relaxation strands or 10.0 ksi for stress-relieved strands (ksi)

The losses due to elastic shortening must then be added to these time-dependent losses to determine the total losses. For non-standard members with unusual dimensions or built using staged segmental construction, the refined method of **LRFD [5.9.5.4]** shall be used. For prestressed box girders time-dependent losses shall be determined using the refined method of **LRFD [5.9.5.4]**.

19.3.2.2.3 Fabrication Losses

Fabrication losses are not considered by the designer, but they affect the design criteria used during design. Anchorage losses which occur during stressing and seating of the prestressed strands vary between 1% and 4%. Losses due to temperature change in the strands during cold weather prestressing are 6% for a 60°F change. The construction specifications permit a 5% difference in the jack pressure and elongation measurement without any adjustment.

19.3.2.3 Service Load

During service load, the member is subjected to the same loads that are present after prestress transfer and losses occur, in addition to the effects of the prestressed I-girder and prestressed box girder load-carrying behavior described in the next two sections.

19.3.2.3.1 Prestressed I-Girder

In the case of a prestressed I-girder, the dead load of the deck and diaphragms are always carried by the basic girder section on a simple span. At strand release, the girder dead load moments are calculated based on the full girder length. For all other loading stages, the girder dead load moments are based on the span length. This is due to the type of construction used (that is, nonshored girders simply spanning from one substructure unit to another for single-span as well as multi-span structures).

The live load plus dynamic load allowance along with any superimposed dead load (curb, parapet or median strip which is placed after the deck concrete has hardened) are carried by the continuous composite section.

WisDOT exception to AASHTO:

The standard pier diaphragm is considered to satisfy the requirements of **LRFD [5.14.1.4.5]** and shall be considered to be fully effective.

In the case of multi-span structures with fully effective diaphragms, the longitudinal distribution of the live load, dynamic load allowance and superimposed dead loads are based on a continuous span structure. This continuity is achieved by:

a. Placing non-prestressed (conventional) reinforcement in the deck area over the interior supports.

b. Casting concrete between and around the abutting ends of adjacent girders to form a diaphragm at the support. Girders shall be in line at interior supports and equal numbers of girders shall be used in adjacent spans. The use of variable numbers of girders between spans requires prior approval by BOS.

If the span length ratio of two adjacent spans exceeds 1.5, the girders are designed as simple spans. In either case, the stirrup spacing is detailed the same as for continuous spans and bar steel is placed over the supports equivalent to continuous span design. It should be noted that this value of 1.5 is not an absolute structural limit.

19.3.2.3.2 Prestressed Box Girder

In the case of prestressed box girders with a thin concrete overlay, the dead load together with the live load and dynamic load allowance are carried by the basic girder section.

When this girder type has a composite section, the dead load of the deck is carried by the basic section and the live load, dynamic load allowance and any superimposed dead loads are carried by the composite section. A composite section shall consist of a reinforced deck, 6” minimum thickness, with composite shear reinforcement extending into the deck.

WisDOT policy item:

The use of prestressed box girders is subject to prior-approval by the Bureau of Structures. These structures are currently limited to the following requirements:

- Single spans
- Composite section details (design and rating based on non-composite section)
- 30 degree maximum skew
- AADT < 3,500 on non-NHS roadways

Variations to these requirements require approval by the Bureau of Structures.

19.3.2.4 Factored Flexural Resistance

At the final stage, the factored flexural resistance of the composite section is considered. Since the member is designed on a service load basis, it must be checked for its factored flexural resistance at the Strength I limit state. See section 17.2.3 for a discussion on limit states.

The need for both service load and strength computations lies with the radical change in a member's behavior when cracks form. Prior to cracking, the gross area of the member is effective. As a crack develops, all the tension in the concrete is picked up by the reinforcement. If the percentage of reinforcement is small, there is very little added capacity between cracking and failure.

19.3.2.5 Fatigue Limit State

At the final stage, the member is checked for the Fatigue I limit state. See section 17.2.3 for a discussion on limit states. Allowable compressive stresses in the concrete and tensile stresses in the non-prestressed reinforcement are checked.

19.3.3 Design Procedure

The intent of this section is to provide the designer with a general outline of steps for the design of pretensioned members. Sections of interest during design include, but are not limited to, the following locations:

- 10^{th} points
- Hold-down points
- Regions where the prestress force changes (consider the effects of transfer and development lengths, as well as the effects of debonded strands)
- Critical section(s) for shear

The designer must consider the amount of prestress force at each design section, taking into account the transfer length and development length, if appropriate.

19.3.3.1 Prestressed I-Girder Member Spacing

A trial prestressed I-girder arrangement is made by using Table 19.3-1 and Table 19.3-2 as a guide. An ideal spacing results in equal strands for interior and exterior girders, together with an optimum slab thickness. Current practice is to use a minimum haunch of (1-1/4" plus deck cross slope times one-half top flange width) for section property calculations and then use a 3" average haunch for concrete preliminary quantity calculations. After preliminary design this value should be revised as needed as outlined in 19.3.4. The maximum slab overhang dimensions are detailed in 17.6.2.

For prestressed I-girder bridges, other than pedestrian or other unusual structures, four or more girders shall be used.

19.3.3.2 Prestressed Box Girder Member Spacing

The prestressed box girder is used in an adjacent multi-beam system only. Precast units are placed side by side and locked (post-tensioned) together. The span length, desired roadway width and live loading control the size of the member.

When selecting a 3' wide section vs. 4' wide section, do not mix 3’ wide and 4’ wide sections across the width of the bridge. Examine the roadway width produced by using all 3’ wide sections or all 4’ wide sections and choose the system that is the closest to but greater than the required roadway width. While 3’ wide sections may produce a slightly narrower roadway width 4’ wide sections are still preferred since they require fewer sections. Verify the required roadway width is possible when considerations are made for the roadway cross-slope. Table 19.3-3 states the approximate span limitations for each section depth. See Standard Detail 19.53 for the number of sections for standard roadway clear widths. Coordinate roadway width with roadway designers and consider some variability.

19.3.3.3 Dead Load

For a detailed discussion of the application of dead load, refer to 17.2.4.1.

The dead load moments and shears due to the girder and concrete deck are computed for simple spans. When superimposed dead loads are considered, the superimposed dead load moments are based on continuous spans.

A superimposed dead load of 20 psf is to be included in all designs which account for a possible future concrete overlay wearing surface. The future wearing surface shall be applied between the faces of curbs or parapets and shall be equally distributed among all the girders in the cross section.

For a cross section without a sidewalk, any curb or parapet dead load is distributed equally to all girders.

For a cross section with a sidewalk and barrier on the overhang, sidewalk and barrier dead loads shall be applied to the exterior girder by the lever rule. These loads shall also be applied to the interior girder by dividing the weight equally among all the girders. A more detailed discussion of dead load distribution can be found in 17.2.8.

19.3.3.4 Live Load

The HL-93 live load shall be used for all new bridges. Refer to section 17.2.4.2 for a detailed description of the HL-93 live load, including the design truck, design tandem, design lane, and double truck.

19.3.3.5 Live Load Distribution

The live load distribution factors shall be computed as specified in **LRFD [4.6.2.2].** Table 17.2-7 summarizes the equations required for prestressed I-girders. The moment and shear distribution factors for prestressed I-girders are determined using equations that consider girder spacing, span length, deck thickness, the number of girders, skew and the longitudinal stiffness parameter. See the WisDOT policy item for live load distribution factors for prestressed box girders.

Separate shear and moment distribution factors are computed for interior and exterior girders. The applicability ranges of the distribution factors shall also be considered. If the applicability ranges are not satisfied, then conservative assumptions must be made based on sound engineering judgment.

WisDOT policy item:

The typical cross section for prestressed box girders shall be type “g” as illustrated in **LRFD [Table 4.6.2.2.1-1]**.

For prestressed box girders, the St. Venant torsional inertia, J, may be calculated as closed thin-walled sections for sections with voids, and as solid sections for sections without voids in accordance with **LRFD [C4.6.2.2.1]**.

See 17.2.8 for additional information regarding live load distribution.

19.3.3.6 Dynamic Load Allowance

The dynamic load allowance, IM, is given by **LRFD [3.6.2]**. Dynamic load allowance equals 33% for all live load limit states except the fatigue limit state and is not applied to pedestrian loads or the lane load portion of the HL-93 live load. See 17.2.4.3 for further information regarding dynamic load allowance.

19.3.3.7 Prestressed I-Girder Deck Design

The design of concrete decks on prestressed I-girders is based on **LRFD [4.6.2.1]**. Moments from truck wheel loads are distributed over a width of deck which spans perpendicular to the girders. This width is known as the distribution width and is given by **LRFD [Table 4.6.2.1.3-1]**. See 17.5 for further information regarding deck design.

19.3.3.8 Composite Section

The effective flange width is the width of the deck slab that is to be taken as effective in composite action for determining resistance for all limit states. The effective flange width, in accordance with **LRFD [4.6.2.6]**, is equal to the tributary width of the girder for interior girders. For exterior girders, it is equal to one half the effective flange width of the adjacent interior girder plus the overhang width. The effective flange width shall be determined for both interior and exterior beams.

Since the deck concrete has a lower strength than the girder concrete, it also has a lower modulus of elasticity. Therefore, when computing composite section properties, the effective flange width (as stated above) must be reduced by the ratio of the modulus of elasticity of the deck concrete divided by the modulus of elasticity of the girder concrete.

WisDOT exception to AASHTO:

WisDOT uses the formulas shown below to determine E_c for prestressed girder design. For 6 ksi girder concrete, E_c is 5,500 ksi, and for 4 ksi deck concrete, E_c is 4,125 ksi. The E_c value of 5,500 ksi for 6 ksi girder concrete strength was determined from deflection studies. These equations are used in place of those presented in **LRFD [5.4.2.4]** for the following calculations: strength, section properties, and deflections due to externally applied dead and live loads.

For slab concrete strength other than 4 ksi, E_c is calculated from the following formula:

$$E_c = \frac{4{,}125\sqrt{f'_c}}{\sqrt{4}} \text{ (ksi)}$$

For girder concrete strengths other than 6 ksi, E_c is calculated from the following formula:

$$E_c = \frac{5{,}500\sqrt{f'_c}}{\sqrt{6}} \text{ (ksi)}$$

WisDOT policy item:

WisDOT uses the equation presented in **LRFD [5.4.2.4]** (and shown below) to calculate the modulus of elasticity at the time of release using the specified value of f'ci. This value of E_i is used for loss calculations and for girder camber due to prestress forces and girder self weight.

$$E_c = 33{,}000 \cdot K_1 \cdot w_c^{1.5}\sqrt{f'_{ci}}$$

Where:

K_1	=	Correction factor for source of aggregate, use 1.0 unless previously approved by BOS.
w_c	=	Unit weight of concrete, 0.150 (kcf)
f'_{ci}	=	Specified compressive strength of concrete at the time of release (ksi)

19.3.3.9 Design Stress

In many cases, stress at the Service III limit state in the bottom fiber at or near midspan after losses will control the flexural design. Determine a trial strand pattern for this condition and proceed with the flexural design, adjusting the strand pattern if necessary.

The design stress is the sum of the Service III limit state bottom fiber stresses due to non-composite dead load on the basic girder section, plus live load, dynamic load allowance and superimposed dead load on the composite section, as follows:

$$f_{des} = \frac{M_{d(nc)}}{S_{b(nc)}} + \frac{M_{d(c)} + M_{(LL+IM)}}{S_{b(c)}}$$

Where:

f_{des}	=	Service III design stress at section (ksi)
$M_{d(nc)}$	=	Service III non-composite dead load moment at section (k-in)
$M_{d(c)}$	=	Service III superimposed dead load moment at section (k-in)
$M_{(LL+IM)}$	=	Service III live load plus dynamic load allowance moment at section (k-in)
$S_{b(nc)}$	=	Non-composite section modulus for bottom of basic beam (in^3)
$S_{b(c)}$	=	Composite section modulus for bottom of basic beam (in^3)

The point of maximum stress is generally 0.5 of the span for both end and intermediate spans. But for longer spans (over 100'), the 0.4 point of the end span may control and should be checked.

19.3.3.10 Prestress Force

With f_{des} known, compute the required effective stress in the prestressing steel after losses, f_{pe}, needed to counteract all the design stress except an amount of tension equal to the tensile stress limit listed in **LRFD [Table 5.9.4.2.2-1]**. The top of the girder is subjected to severe corrosion conditions and the bottom of the girder is subjected to moderate exposure. The Service III tensile stress at the bottom fiber after losses for pretensioned concrete shall not exceed $0.19\sqrt{f'_c}$ (or 0.6 ksi). Therefore:

$$f_{pe} = f_{des} - \min(0.19\sqrt{f'_c} \text{ or } 0.6 \text{ ksi})$$

Note: A conservative approach used in hand calculations is to assume that the allowable tensile stress equals zero.

Applying the theory discussed in 19.2:

$$f_{pe} = \frac{P_{pe}}{A}\left(1 + \frac{ey}{r^2}\right)$$

Where:

P_{pe}	=	Effective prestress force after losses (kips)
A	=	Basic beam area (in^2)
e	=	Eccentricity of prestressing strands with respect to the centroid of the basic beam at section (in)
r	=	$\sqrt{\frac{I}{A}}$ of the basic beam (in)

For prestressed box girders, assume an e and apply this to the above equation to determine P_{pe} and the approximate number of strands. Then a trial strand pattern is established using the Standard Details as a guide, and a check is made on the assumed eccentricity. For prestressed I-girders, f_{pe} is solved for several predetermined patterns and is tabulated in the Standard Details.

Present practice is to detail all spans of equal length with the same number of strands, unless a span requires more than three additional strands. In this case, the different strand arrangements are detailed along with a plan note stating: "The manufacturer may furnish all girders with the greater number of strands."

19.3.3.11 Service Limit State

Several checks need to be performed at the service limit state. Refer to the previous narrative in 19.3.3 for sections to be investigated and section 17.2.3.2 for discussion on the service limit state. Note that Service I limit state is used when checking compressive stresses and Service III limit state is used when checking tensile stresses.

The following should be verified by the engineer:

- Verify that the Service III tensile stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed the limits presented in **LRFD [Table 5.9.4.1.2-1]**, which depend upon whether or not the strands are bonded and satisfy stress requirements. This will generally control at the top of the beam near the beam ends where the dead load moment approaches zero and is not able to counter the tensile stress at the top of the beam induced by the prestress force. When the calculated tensile stress exceeds the stress limits, the strand pattern must be modified by draping or partially debonding the strand configuration.

- Verify that the Service I compressive stress due to beam self-weight and prestress applied to the basic beam at transfer does not exceed 0.60 f'_{ci}, as presented in **LRFD [5.9.4.1.1]**. This will generally control at the bottom of the beam near the beam ends or at the hold-down point if using draped strands.

- Verify that the Service III tensile stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.4.2.2-1]**. No tensile stress shall be permitted for unbonded strands. The tensile stress of bonded strands shall not exceed $0.19\sqrt{f'_c}$ (or 0.6 ksi) as all strands shall be considered to be in moderate corrosive conditions. This will generally control at the bottom of the beam near midspan and at the top of the continuous end of the beam.

- Verify that the Service I compressive stress due to all dead and live loads applied to the appropriate sections after losses does not exceed the limits presented in **LRFD [Table 5.9.4.2.1-1]**. Two checks need to be made for girder bridges. The compressive stress due to the sum of effective prestress and permanent loads shall not exceed 0.45 f'_c (ksi). The compressive stress due to the sum of effective prestress, permanent loads and transient loads shall not exceed $0.60\phi_w f'_c$ (ksi). The term ϕ_w, a reduction factor applied to thin-walled box girders, shall be 1.0 for WisDOT standard girders.

- Verify that Fatigue I compressive stress due to fatigue live load and one-half the sum of effective prestress and permanent loads does not exceed 0.40 f'_c (ksi) **LRFD [5.5.3.1]**.

- Verify that the Service I compressive stress at the top of the deck due to all dead and live loads applied to the appropriate sections after losses does not exceed 0.40 f'_c.

WisDOT policy item:

The top of the prestressed I-girders at interior supports shall be designed as reinforced concrete members at the strength limit state in accordance with **LRFD [5.14.1.4.6]**. In this case, the stress limits for the service limit state shall not apply to this region of the precast girder.

19.3.3.12 Raised, Draped or Partially Debonded Strands

When straight strands are bonded for the full length of a prestressed girder, the tensile and compressive stresses near the ends of the girder will likely exceed the allowable service limit state stresses. This occurs because the strand pattern is designed for stresses at or near midspan, where the dead load moment is highest and best able to balance the effects of the prestress. Near the ends of the girder this dead load moment approaches zero and is less able to balance the prestress force. This results in tensile stresses in the top of the girder and compressive stresses in the bottom of the girder. The allowable initial tensile and compressive stresses are presented in the first two bullet points of 19.3.3.11. These stresses are a function of f'_{ci}, the compressive strength of concrete at the time of prestress force transfer. Transfer and development lengths should be considered when checking stresses near the ends of the girder.

The designer should start with a straight (raised), fully bonded strand pattern. If this overstresses the girder near the ends, the following methods shall be utilized to bring the girder within the allowable stresses. These methods are listed in order of preference and discussed in the following sections:

1. Use raised strand pattern (If excessive top flange reinforcement or if four or more additional strands versus a draped strand pattern are required, consider the draped strand alternative)

2. Use draped strand pattern

3. Use partially debonded strand pattern (to be used sparingly)

Only show one strand pattern per span (i.e. Do not show both raised and draped span alternatives for a given span).

A different girder spacing may need to be selected. It is often more economical to add an extra girder line than to maximize the number of strands and use debonding.

Prestressed box girders strands are to be straight, bonded, and located as shown in the Standard Details.

19.3.3.12.1 Raised Strand Patterns

Some of the standard strand patterns listed in the Standard Details show a raised strand pattern. Generally strands are placed so that the center of gravity of the strand pattern is as close as possible to the bottom of the girder. With a raised strand pattern, the center of gravity of the strand pattern is raised slightly and is a constant distance from the bottom of the girder for its entire length. Present practice is to show a standard raised arrangement as a preferred alternate to draping for short spans. For longer spans, debonding at the ends of the strands is an alternate (see 19.3.3.12.3). Use 0.6" strands for all raised patterns.

19.3.3.12.2 Draped Strand Patterns

Draping some of the strands is another available method to decrease stresses from prestress at the ends of the I-beam where the stress due to applied loads are minimum.

The typical strand profile for this technique is shown in Figure 19.3-1.

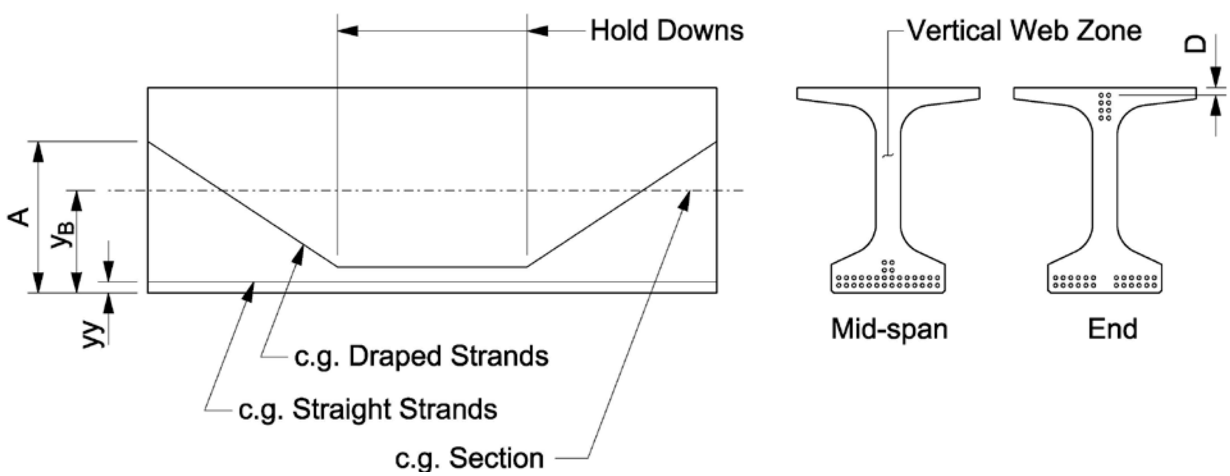

Figure 19.3-1
Typical Draped Strand Profile

Note that all the strands that lie within the "vertical web zone" of the mid-span arrangement are used in the draped group.

The engineer should show only one strand size for the draped pattern on the plans. Use only 0.5" strands for the draped pattern on 28" and 36" prestressed I-girders and 0.6" strands for all raised (straight) patterns for these shapes. Use 0.6" strands, only, for 36W", 45W", 54W", 72W" and 82W" prestressed I-girders. See Chapter 40 standards for 45", 54" and 70" prestressed I-girders.

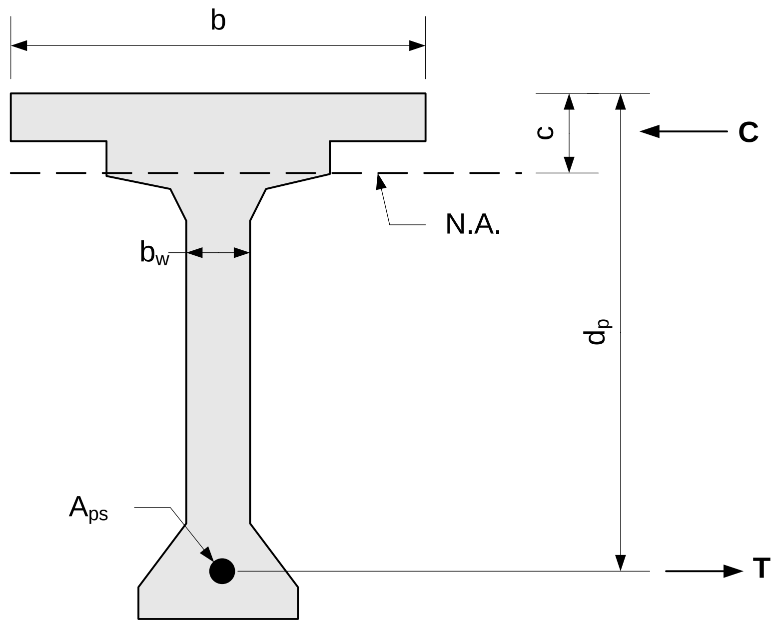

Figure 19.3-3
Depth to Neutral Axis, c

Verify that rectangular section behavior is allowed by checking that the depth of the equivalent stress block, a, is less than or equal to the structural deck thickness. If it is not, then T-section behavior provisions should be followed. If the T-section provisions are used, the compression block will be composed of two different materials with different compressive strengths. In this situation, **LRFD [C5.7.2.2]** recommends using β_1 and α_1 corresponding to the lower f'_c. The following equation for c shall be used for T-section behavior: **LRFD [5.7.3.1.1]**

$$c = \frac{A_{ps} f_{pu} - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

Where:

b_w = Width of web (in) – use the top flange width if the compression block does not extend below the haunch.

h_f = Depth of compression flange (in)

The factored flexural resistance presented in **LRFD [5.7.3.2.2]** is simplified by neglecting the area of mild compression and tension reinforcement. Furthermore, if rectangular section behavior is allowed, then b_w = b, where b_w is the web width as shown in Figure 19.3-3. The equation then reduces to:

$$M_r = \phi A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right)$$

Where:

M_r	=	Factored flexural resistance (kip-in)
ϕ	=	Resistance factor
f_{ps}	=	Average stress in prestressing steel at nominal bending resistance (refer to **LRFD [5.7.3.1.1]**) (ksi)

If the T-section provisions must be used, the factored moment resistance equation is then:

$$M_r = \phi A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right) + \alpha_1 \phi f'_c \left(b - b_w \right) h_f \left(\frac{a}{2} - \frac{h_f}{2} \right)$$

Where:

h_f	=	Depth of compression flange with width, b (in)

The engineer must then verify that M_r is greater than or equal to M_u.

<u>WisDOT exception to AASHTO:</u>

WisDOT standard prestressed I-girders and strand patterns are tension-controlled. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required when the standard girders and strand patterns are used, and $\phi = 1$.

19.3.3.13.2 Minimum Reinforcement

Per **LRFD [5.7.3.3.2]**, the minimum amount of prestressed reinforcement provided shall be adequate to develop an M_r at least equal to the lesser of M_{cr}, or $1.33M_u$.

M_{cr} is the cracking moment, and is given by:

$$M_{cr} = \gamma_3 \left[S_c \left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) - 12M_{dnc} \left[(S_c/S_{nc}) - 1 \right] \right]$$

The factored horizontal interface shear shall then be determined as:

$$V_{ui} = 12 v_{ui} b_{vi}$$

The nominal interface shear resistance shall be taken as:

$$V_{ni} = cA_{cv} + \mu \left[A_{vf} f_y + P_c \right]$$

Where:

A_{cv}	=	Concrete area considered to be engaged in interface shear transfer. This value shall be set equal to $12b_{vi}$ (ksi)
c	=	Cohesion factor specified in **LRFD [5.8.4.3]**. This value shall be taken as 0.28 ksi for WisDOT standard girders with a cast-in-place deck
μ	=	Friction factor specified in **LRFD [5.8.4.3]**. This value shall be taken as 1.0 for WisDOT standard girders with a cast-in-place deck (dim.)
A_{vf}	=	Area of interface shear reinforcement crossing the shear plan within the area A_{cv} (in^2)
f_y	=	Yield stress of shear interface reinforcement not to exceed 60 (ksi)
P_c	=	Permanent net compressive force normal to the shear plane (kips)

P_c shall include the weight of the deck, haunch, parapets, and future wearing surface. A conservative assumption that may be considered is to set $P_c = 0.0$.

The nominal interface shear resistance, V_{ni}, shall not exceed the lesser of:

$$V_{ni} \le K_1 f'_c A_{cv} \quad \text{or} \quad V_{ni} \le K_2 A_{cv}$$

Where:

K_1	=	Fraction of concrete strength available to resist interface shear as specified in **LRFD [5.8.4.3].** This value shall be taken as 0.3 for WisDOT standard girders with a cast-in-place deck (dim.)
K_2	=	Limiting interface shear resistance as specified in **LRFD [5.8.4.3].** This value shall be taken as 1.8 ksi for WisDOT standard girders with a cast-in-place deck

WisDOT policy item:

The stirrups that extend into the deck slab presented on the Standards are considered adequate to satisfy the minimum reinforcement requirements of **LRFD [5.8.4.4]**

19.3.3.16 Web Shear Reinforcement

Web shear reinforcement consists of placing conventional reinforcement perpendicular to the axis of the girder.

WisDOT policy item:

Web shear reinforcement shall be designed by **LRFD [5.8.3.4.3]** (Simplified Procedure) using the Strength I limit state for WisDOT standard girders.

WisDOT prefers girders with spacing symmetrical about the midspan in order to simplify design and fabrication. The designer is encouraged to simplify the stirrup arrangement as much as possible. For vertical stirrups, the required area of web shear reinforcement is given by the following equation:

$$A_v \geq \frac{(V_n - V_c)s}{f_y d_v \cot\theta} \qquad \left(\text{or } 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} \quad \text{minimum}\right)$$

Where:

A_v	=	Area of transverse reinforcement within distance, s (in^2)
V_n	=	Nominal shear resistance (kips)
V_c	=	Nominal shear resistance provided by tensile stress in the concrete (kips)
s	=	Spacing of transverse reinforcement (in)
f_y	=	Specified minimum yield strength of transverse reinforcement (ksi)
d_v	=	Effective shear depth as determined in **LRFD [5.8.2.9]** (in)
b_v	=	Minimum web width within depth, d_v

$\cot\theta$ shall be taken as follows:

WisDOT policy item:

Based on past performance, for prestressed I-girders the upper limit for web reinforcement spacing, s_{max}, per **LRFD [5.8.2.7]** will be reduced to 18 inches.

When determining shear reinforcement, spacing requirements as determined by analysis at 1/10th points, for example, should be carried-out to the next 1/10th point. As an illustration, spacing requirements for the 1/10th point should be carried out to very close to the 2/10th point, as the engineer, without a more refined analysis, does not know what the spacing requirements would be at the 0.19 point. For the relatively small price of stirrups, don't shortchange the shear capacity of the prestressed girder.

The web reinforcement spacing shall not exceed the maximum permitted spacing determined as:

- If $\upsilon_u < 0.125f'_c$, then $s_{max} = 0.8d_v \leq 18"$
- If $\upsilon_u \geq 0.125f'_c$, then $s_{max} = 0.4d_v \leq 12"$

Where:

$$\upsilon_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ per **LRFD [5.8.2.9]**.

The nominal shear resistance, $V_c + V_s$, is limited by the following:

$$V_c + \frac{A_v f_y d_v \cot\theta}{s} \leq 0.25f'_c b_v d_v$$

Reinforcement in the form of vertical stirrups is required at the extreme ends of the girder. The stirrups are designed to resist 4% of the total prestressing force at transfer at a unit stress of 20 ksi and are placed within h/4 of the girder end, where h is the total girder depth. For a distance of 1.5d from the ends of the beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall be shaped to enclose the strands, shall be a #3 bar or greater and shall be spaced at less than or equal to 6”. Note that the reinforcement shown on the Standard Details sheets satisfies these requirements.

Welded wire fabric may be used for the vertical reinforcement. It must be deformed wire with a minimum size of D18.

Per **LRFD [5.8.3.5]**, at the inside edge of the bearing area to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi} - 0.5 V_s \right) \cot\theta$$

In the above equation, $\cot\theta$ is as defined in the V_c discussion above, and V_s is the shear reinforcement resistance at the section considered. Any lack of full reinforcement development shall be accounted for. Note that the reinforcement shown on the Standard Detail sheets satisfies these requirements.

19.3.3.17 Continuity Reinforcement

The design of non-prestressed reinforcement for negative moment at the support is based on the Strength I limit state requirements of **LRFD [5.7.3]**:

$$M_u = 1.25DC + 1.50DW + 1.75(LL + IM)$$

LRFD [5.5.4.2] allows a ϕ factor equal to 0.9 for tension-controlled reinforced concrete sections such as the bridge deck.

The continuity reinforcement consists of mild steel reinforcement in the deck in the negative moment region over the pier. Consider both the non-composite and the superimposed dead loads and live loads for the Strength I design of the continuity reinforcement in the deck.

Moment resistance is developed in the same manner as shown in 19.3.3.13.1 for positive moments, except that the bottom girder flange is in compression and the deck is in tension. The moment resistance is formed by the couple resulting from the compression force in the bottom flange and the tension force from the longitudinal deck steel. Consider A_s to consist of the longitudinal deck steel present in the deck slab effective flange width as determined in 19.3.3.8. The distance, d_p, is taken from the bottom of the girder flange to the center of the longitudinal deck steel.

<u>WisDOT exception to AASHTO:</u>

Composite sections formed by WisDOT standard prestressed I-girders shall be considered to be tension-controlled for the design of the continuity reinforcement. The ε_t check, as specified in **LRFD [5.7.2.1]**, is not required, and $\phi = 0.9$.

<u>WisDOT policy item:</u>

New bridge designs shall consider only the top mat of longitudinal deck steel when computing the continuity reinforcement capacity.

WisDOT policy item:

The continuity reinforcement shall be based on the greater of either the interior girder design or exterior girder and detailed as typical reinforcement for the entire width of the bridge deck. However, do not design the continuity steel based on the exterior girder design beneath a raised sidewalk. The continuity steel beneath a raised sidewalk should not be used for rating.

Based on the location of the neutral axis, the bottom flange compressive force may behave as either a rectangle or a T-section. On WisDOT standard prestressed I-girders, if the depth of the compression block, a, falls within the varying width of the bottom flange, the compression block acts as an idealized T-section. In this case, the width, b, shall be taken as the bottom flange width, and the width, b_w, shall be taken as the bottom flange width at the depth "a". During T-section behavior, the depth, h_f, shall be taken as the depth of the bottom flange of full width, b. See Figure 19.3-4 for details. Ensure that the deck steel is adequate to satisfy $M_r \geq M_u$.

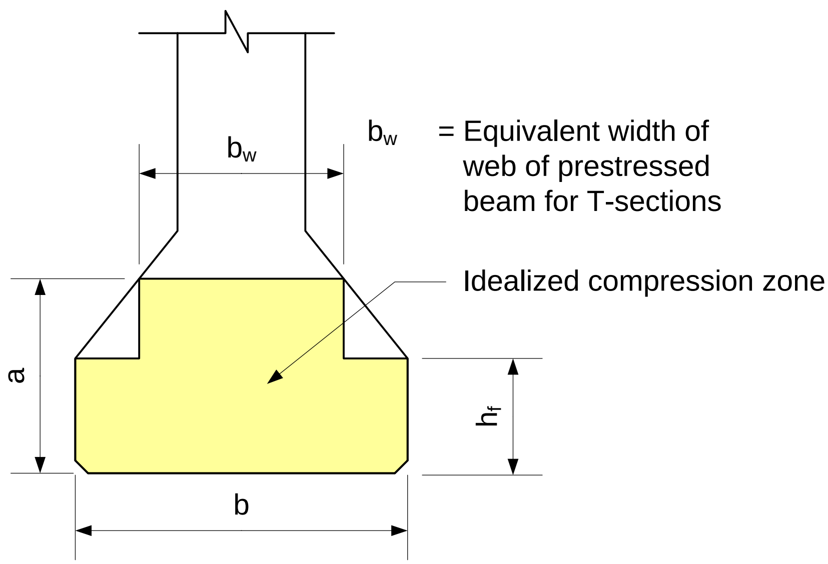

Figure 19.3-4
T-Section Compression Flange Behavior

The continuity reinforcement should also be checked to ensure that it meets the crack control provisions of **LRFD [5.7.3.4]**. This check shall be performed assuming severe exposure conditions. Only the superimposed loads shall be considered for the Service and Fatigue requirements.

The concrete between the abutting girder ends is usually of a much lesser strength than that of the girders. However, tests[1] have shown that, due to lateral confinement of the diaphragm concrete, the girder itself fails in ultimate negative compression rather than failure in the material between its ends. Therefore the ultimate compressive stress, f'_c, of the girder concrete is used in place of that of the diaphragm concrete.

This assumption has only a slight effect on the computed amount of reinforcement, but it has a significant effect on keeping the compression force within the bottom flange.

The continuity reinforcement shall conform to the Fatigue provisions of **LRFD [5.5.3]**.

The transverse spacing of the continuity reinforcement is usually taken as the whole or fractional spacing of the D bars as given in 17.5.3.2. Grade 60 bar steel is used for continuity reinforcement. Required development lengths for deformed bars are given in Chapter 9 – Materials.

WisDOT exception to AASHTO:

The continuity reinforcement is not required to be anchored in regions of the slab that are in compression at the strength limit state as stated in **LRFD [5.14.1.4.8]**. The following locations shall be used as the cut off points for the continuity reinforcement:

1. When ½ the bars satisfy the Strength I moment envelope (considering both the non-composite and composite loads) as well as the Service and Fatigue moment envelopes (considering only the composite moments), terminate ½ of the bars. Extend these bars past this cutoff point a distance not less than the girder depth or 1/16 the clear span for embedment length requirements.

2. Terminate the remaining one-half of the bars an embedment length beyond the point of inflection. The inflection point shall be located by placing a 1 klf load on the composite structure. This cut-off point shall be at least 1/20 of the span length or 4' from point 1, whichever is greater.

Certain secondary features result when spans are made continuous. That is, positive moments develop over piers due to creep[5], shrinkage and the effects of live load and dynamic load allowance in remote spans. The latter only exists for bridges with three or more spans.

These positive moments are somewhat counteracted by negative moments resulting from differential shrinkage[4] between the cast-in-place deck and precast girders along with negative moments due to superimposed dead loads. However, recent field observations cited in **LRFD [C5.14.1.4.2]** suggest that these moments are less than predicted by analysis. Therefore, negative moments caused by differential shrinkage should be ignored in design.

WisDOT exception to AASHTO:

WisDOT requires the use of a negative moment connection only. The details for a positive moment connection per **LRFD [5.14.1.4]** are not compatible with the Standard Details and should not be provided.

19.3.3.18 Camber and Deflection

The prestress camber and dead load deflection are used to establish the vertical position of the deck forms with respect to the girder. The theory presented in the following sections apply to a narrow set of circumstances. The designer is responsible for ensuring that the theoretical camber accounts for the loads applied to the girder. For example, if the diaphragms of a prestressed I-girder are configured so there is one at each of the third points instead of one at midspan, the term in the equation for $\Delta_{nc(DL)}$ related to the diaphragms in 19.3.3.18.2 would need to be modified to account for two point loads applied at the third points instead of one point load applied at midspan.

Deflection effects due to individual loads may be calculated separately and superimposed, as shown in this section. The *PCI Design Handbook* provides design aids to assist the designer in the evaluation of camber and deflection, including cambers for prestress forces and loads, and beam design equations and diagrams.

Figure 19.3-5 illustrates a typical prestressed I-girder with a draped strand profile.

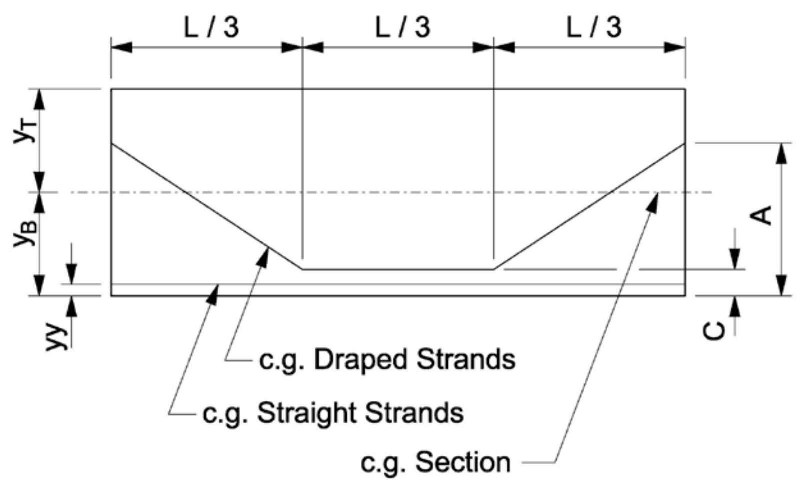

Figure 19.3-5
Typical Draped Strand Profile

19.3.3.18.1 Prestress Camber

The prestressing strands produce moments in the girder as a result of their eccentricity and draped pattern. These moments induce a camber in the girder. The values of the camber are calculated as follows:

Eccentric straight strands induce a constant moment of:

$$M_1 = \frac{1}{12}\left(P_i^s(y_B - yy)\right)$$

Where:

M_1	=	Moment due to initial prestress force in the straight strands minus the elastic shortening loss (k-ft)
P_i^s	=	Initial prestress force in the straight strands minus the elastic shortening loss (kips)
y_B	=	Distance from center of gravity of beam to bottom of beam (in)
yy	=	Distance from center of gravity of straight strands to bottom of beam (in)

This moment produces an upward deflection at midspan which is given by:

$$\Delta_s = \frac{M_1 L^2}{8E_i I_b} \quad \text{(with all units in inches and kips)}$$

For moments expressed in kip-feet and span lengths expressed in feet, this equation becomes the following:

$$\Delta_s = \frac{M_1 L^2}{8E_i I_b}\left(\frac{12}{1}\right)\left(\frac{12^2}{1}\right) = \frac{M_1 L^2}{8E_i I_b}\left(\frac{1728}{1}\right)$$

$$\Delta_s = \frac{216 M_1 L^2}{E_i I_b} \quad \text{(with units as shown below)}$$

Where:

Δ_s	=	Deflection due to force in the straight strands minus elastic shortening loss (in)
L	=	Span length between centerlines of bearing (ft)
E_i	=	Modulus of elasticity at the time of release (see 19.3.3.8) (ksi)
I_b	=	Moment of inertia of basic beam (in^4)

The draped strands induce the following moments at the ends and within the span:

$$M_2 = \frac{1}{12}\left(P_i^D\left(A - C\right)\right)$$, which produces upward deflection, and

$$M_3 = \frac{1}{12}\left(P_i^D\left(A - y_B\right)\right)$$, which produces downward deflection when A is greater than y_B

Where:

M_2, M_3	=	Components of moment due to initial prestress force in the draped strands minus the elastic shortening loss (k-ft)
P_i^D	=	Initial prestress force in the draped strands minus the elastic shortening loss (kips)
A	=	Distance from bottom of beam to center of gravity of draped strands at centerline of bearing (in)
C	=	Distance from bottom of beam to center of gravity of draped strands between hold-down points (in)

These moments produce a net upward deflection at midspan, which is given by:

$$\Delta_D = \frac{216L^2}{E_i I_b}\left(\frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_D	=	Deflection due to force in the draped strands minus elastic shortening loss (in)

The combined upward deflection due to prestress is:

$$\Delta_{PS} = \Delta_s + \Delta_D = \frac{216L^2}{E_i I_b}\left(M_1 + \frac{23}{27}M_2 - M_3\right)$$

Where:

Δ_{PS}	=	Deflection due to straight and draped strands (in)

The downward deflection due to beam self-weight at release is:

$$\Delta_{o(DL)} = \frac{5W_bL^4}{384E_iI_b} \quad \text{(with all units in inches and kips)}$$

Using unit weights in kip per foot, span lengths in feet, E in ksi and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_bL^4}{384E_iI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) = \frac{5W_bL^4}{384E_iI_b}\left(\frac{20736}{12}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{E_iI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{o(DL)}$	=	Deflection due to beam self-weight at release (in)
W_b	=	Beam weight per unit length (k/ft)

Therefore, the anticipated prestress camber at release is given by:

$$\Delta_i = \Delta_{PS} - \Delta_{o(DL)}$$

Where:

Δ_i	=	Prestress camber at release (in)

Camber, however, continues to grow after the initial strand release. For determining substructure beam seats, average concrete haunch values (used for both DL and quantity calculations) and the required projection of the vertical reinforcement from the tops of the prestressed girders, **a camber multiplier of 1.4 shall be used**. This value is multiplied by the theoretical camber at release value.

19.3.3.18.2 Dead Load Deflection

The downward deflection of a prestressed I-girder due to the dead load of the deck and a midspan diaphragm is:

$$\Delta_{nc(DL)} = \frac{5W_{deck}L^4}{384EI_b} + \frac{P_{dia}L^3}{48EI_b} \quad \text{(with all units in inches and kips)}$$

Using span lengths in units of feet, unit weights in kips per foot, E in ksi, and I_b in inches^4, this equation becomes the following:

$$\Delta_s = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{1}{12}\right)\left(\frac{12^4}{1}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{12^3}{1}\right) = \frac{5W_{deck}L^4}{384EI_b}\left(\frac{20736}{12}\right) + \frac{P_{dia}L^3}{48EI_b}\left(\frac{1728}{1}\right)$$

$$\Delta_{o(DL)} = \frac{22.5W_bL^4}{EI_b} + \frac{36P_{dia}L^3}{EI_b} \quad \text{(with units as shown below)}$$

Where:

$\Delta_{nc(DL)}$	=	Deflection due to non-composite dead load (deck and midspan diaphragm) (in)
W_{deck}	=	Deck weight per unit length (k/ft)
P_{dia}	=	Midspan diaphragm weight (kips)
E	=	Girder modulus of elasticity at final condition (see 19.3.3.8) (ksi)

A similar calculation is done for parapet and sidewalk loads on the composite section. Provisions for deflections due to future wearing surface shall not be included.

For girder structures with raised sidewalks, loads shall be distributed as specified in Chapter 17, and separate deflection calculations shall be performed for the interior and exterior girders.

19.3.3.18.3 Residual Camber

Residual camber is the camber that remains after the prestress camber has been reduced by the composite and non-composite dead load deflection. Residual camber is computed as follows:

$$RC = \Delta_i - \Delta_{nc(DL)} - \Delta_{c(DL)}$$

19.3.4 Prestressed I-Girder Deck Forming

Deck forming requires computing the relationship between the top of girder and bottom of deck necessary to achieve the desired vertical roadway alignment. Current practice for design is to use a minimum haunch of 2" at the edge of the girder flange. This haunch value is also used for calculating composite section properties. This will facilitate current deck forming practices which use 1/2" removable hangers and 3/4" plywood, and it will allow for variations in prestress camber. Also, future deck removal will be less likely to damage the top girder flanges. An average haunch height of 3 inches minimum can be used for determining haunch weight for preliminary design. It should be noted that the actual haunch values should be compared with the estimated values during final design. If there are significant differences in these values, the design should be revised. The actual average haunch height should be used to calculate the concrete quantity reported on the plans as well as the value reported on the prestressed girder details sheet. The actual haunch values at the girder ends shall be used for determining beam seat elevations.

For designs involving vertical curves, Figure 19.3-6 shows two different cases.

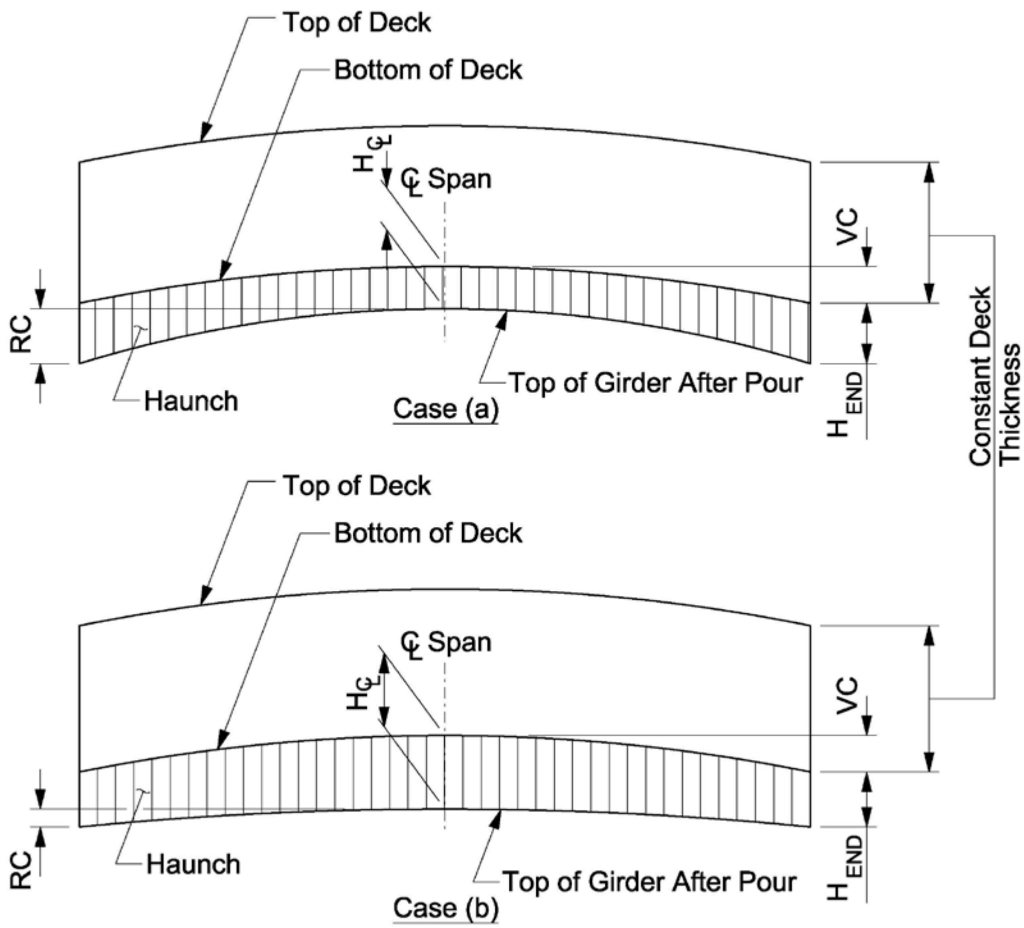

Figure 19.3-6
Relationship Between Top of Girder and Bottom of Deck

In Case (a), VC is less than the computed residual camber, RC, and the minimum haunch occurs at midspan. In Case (b), VC is greater than RC and the minimum haunch occurs at the girder ends.

Deck forms are set to accommodate the difference between the bottom of the deck and the top of the girder under all dead loads placed at the time of construction, including the wet deck concrete and superimposed parapet and sidewalk loads. The deflection of superimposed future wearing surface and live loads are not included.

19.3.4.1 Equal-Span Continuous Structures

For equal-span continuous structures having all spans on the same vertical alignment, the deck forming is the same for each span. This is due to the constant change of slope of the vertical curve or tangent and the same RC per span.

The following equation is derived from Figure 19.3-6:

$$+H_{END} = RC - VC + (+H_{CL})$$

Where:

H_{END}	=	See Figure 19.3-6 (in)
RC	=	Residual camber, positive for upward (in)
VC	=	Difference in vertical curve, positive for crest vertical curves and negative for sag vertical curves (in)
H_{CL}	=	See Figure 19.3-6 (in)

19.3.4.2 Unequal Spans or Curve Combined With Tangent

For unequal spans or when some spans are on a vertical curve and others are on a tangent, a different approach is required. Generally the longer span or the one off the curve dictates the haunch required at the common support. Therefore, it is necessary to pivot the girder about its midspan in order to achieve an equal condition at the common support. This is done mathematically by adding together the equation for each end (abutment and pier), as follows:

$$(+H_{LT}) + (+H_{RT}) = 2[RC - VC + (+H_{CL})]$$

Where:

H_{LT}	=	H_{END} at left (in)
H_{RT}	=	H_{END} at right (in)

With the condition at one end known due to the adjacent span, the condition at the other end is computed.

19.3.5 Construction Joints

The transverse construction joints should be located in the deck midway between the cut-off points of the continuity reinforcement or at the 0.75 point of the span, whichever is closest to the pier. The construction joint should be located at least 1' from the cut-off points.

This criteria keeps stresses in the slab reinforcement due to slab dead load at a minimum and makes deflections from slab dead load closer to the theoretical value.

19.3.6 Strand Types

Low relaxation strands (0.5” and 0.6” in diameter) are currently used in prestressed I-girder and prestressed box girder designs and are shown on the plans. Strand patterns and initial prestressing forces are given on the plans, and deflection data is also shown.

19.3.7 Construction Dimensional Tolerances

Refer to the *AASHTO LRFD Bridge Construction Specifications* for the required dimensional tolerances.

19.3.8 Prestressed I-Girder Sections

WisDOT BOS employs two prestress I-girder section families. One I section family follows the AASHTO standard section, while the other section family follows a wide flange bulb-tee, see Figure 19.3-7. These sections employ draped strand patterns with undraped alternates where feasible. Undraped strand patterns, when practical, should be specified on the designs. For these sections, the cost of draping far exceeds savings in strands. See the Standard Details for the prestressed I-girder sections’ draped and undraped strand patterns. Note, for the 28” prestressed I-girder section the 16 and 18 strand patterns require bond breakers.

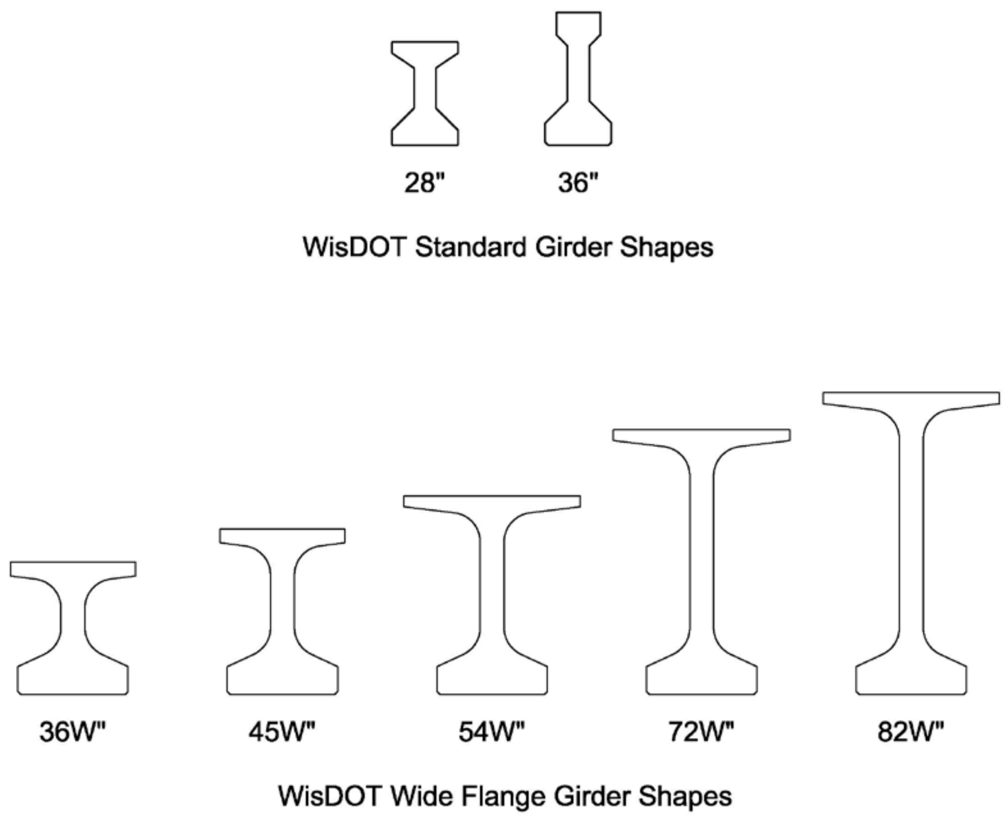

Figure 19.3-7
Prestressed I-Girder Family Details

Table 19.3-1 and Table 19.3-2 provide span lengths versus interior girder spacings for HL-93 live loading on single-span and multiple-span structures for prestressed I-girder sections. Girder spacings are based on using low relaxation strands at $0.75f_{pu}$, concrete haunch thicknesses, slab thicknesses from Chapter 17 – Superstructures - General and a future wearing surface. For these tables, a line load of 0.300 klf is applied to the girder to account for superimposed dead loads.

Several girder shapes have been retired from standard use on new structures. These include the following sizes; 45-inch, 54-inch and 70-inch. These girder shapes are used for girder replacements, widening and for curved new structures where the wide flange sections are not practical. See Chapter 40 – Bridge Rehabilitation for additional information on these girder shapes.

Due to the wide flanges on the 54W, 72W and 82W and the variability of residual camber, haunch heights frequently exceed 2”. An average haunch of 4” was used for all wide flange girders in the following tables. **Do not push the span limits/girder spacing during preliminary design.** See Table 19.3-2 for guidance regarding use of excessively long prestressed I-girders.

Tables are based on:

- Interior prestressed I-girders, 0.5” or 0.6” dia. strands (in accordance with the Standard Details).
- f'_c girder = 8,000 psi
- f'_c slab = 4,000 psi
- Haunch height (dead load) = 2 ½” for 28” and 36” girders

  = 4” for 45W”,54W”,72W” and 82W” girders
- Haunch height (section properties) = 2”
- Required f'_c girder at initial prestress < 6,800 psi

28" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	54	60
6’-6”	54	58
7’-0”	52	56
7’-6”	50	54
8’-0”	50	54
8’-6”	48	52
9’-0”	48	50
9’-6”	46	50
10’-0”	44	48
10’-6”	44	48
11’-0”	42	46
11’-6”	42	46
12’-0”	42	44

36" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	72	78
6’-6”	70	76
7’-0”	70	74
7’-6”	68	72
8’-0”	66	70
8’-6”	64	68
9’-0”	62	68
9’-6”	60	64
10’-0”	60	64
10’-6”	58	62
11’-0”	50	60
11’-6”	50	60
12’-0”	48	58

36W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	94	101
6’-6”	92	99
7’-0”	88	97
7’-6”	87	95
8’-0”	85	93
8’-6”	83	91
9’-0”	82	88
9’-6”	80	86
10’-0”	77	84
10’-6”	76	83
11’-0”	74	81
11’-6”	73	78
12’-0”	71	76

45W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	110	120
6’-6”	109	117
7’-0”	107	115
7’-6”	103	113
8’-0”	101	111
8’-6”	99	108
9’-0”	97	104
9’-6”	95	102
10’-0”	92	100
10’-6”	90	98
11’-0”	88	96
11’-6”	87	93
12’-0”	85	91

Table 19.3-1
Maximum Span Length vs. Girder Spacing

54W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	125	134
6’-6”	123	132
7’-0”	120	129
7’-6”	118	127
8’-0”	116	125
8’-6”	114	122
9’-0”	112	120
9’-6”	110	117
10’-0”	108	115
10’-6”	106	114
11’-0”	102	111
11’-6”	101	109
12’-0”	99	107

72W" Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	153*	164*⊗
6’-6”	150	161*⊗
7’-0”	148	158*
7’-6”	145	156*
8’-0”	143	153*
8’-6”	140	150
9’-0”	138	148
9’-6”	135	144
10’-0”	133	142
10’-6”	131	140
11’-0”	129	137
11’-6”	127	135
12’-0”	124	132

82W” Prestressed I-Girder		
Girder Spacing	Single Span	2 Equal Spans
6’-0”	166*⊗	177*⊗
6’-6”	163*⊗	174*⊗
7’-0”	161*⊗	172*⊗
7’-6”	158*	169*⊗
8’-0”	156*	166*⊗
8’-6”	152	163*⊗
9’-0”	150	160*⊗
9’-6”	147	157*
10’-0”	145	154*
10’-6”	143	152
11’-0”	140	149
11’-6”	136	147
12’-0”	133	144

Table 19.3-2
Maximum Span Length vs. Girder Spacing

* For lateral stability during lifting these girder lengths may require pick-up point locations greater than distance d (girder depth) from the ends of the girder. The designer shall assume that the pick-up points will be at the 1/10 points from the end of the girder and provide extra non-prestressed steel in the top flange, if required, and check the concrete strength near the lift location based on f'_{ci}. A note should be placed on the girder details sheet to reflect that the girder was analyzed for a potential lift at the 1/10 point.

⊗ Due to difficulty manufacturing, transporting and erecting excessively long prestressed girders, consideration should be given to utilizing an extra pier to minimize use of such girders. Approval from the Bureau of Structures is required to utilize any girder over 158 ft. long. (Currently, there is still a moratorium on the use of all 82W"). Steel girders may be considered if the number of piers can be reduced enough to offset the higher costs associated with a steel superstructure.

19.3.8.1 Prestressed I-Girder Standard Strand Patterns

The standard strand patterns presented in the Standard Details were developed to eliminate some of the trial and error involved in the strand pattern selection process. These standard strand patterns should be used whenever possible, with a straight strand arrangement preferred over a draped strand arrangement. The designer is responsible for ensuring that the selected strand pattern meets all LRFD requirements.

Section 19.3.3 discusses the key parts of the design procedure, and how to effectively use the standard strand patterns along with Table 19.3-1 and Table 19.3-2.

The amount of drape allowed is controlled by the girder size and the 2" clearance from center of strand to top of girder. See the appropriate Standard Girder Details for guidance on draping.

19.3.9 Prestressed Box Girders Post-Tensioned Transversely

These sections may be used for skews up to 30° with the transverse post-tensioning ducts placed along the skew. Skews over 30° are not recommended, but if absolutely required the transverse post-tensioning ducts should be placed perpendicular to the prestressed sections. Also for skews over 30° a more refined method of analysis should be used such as an equivalent plate analysis or a finite element analysis.

Details for transverse post-tensioning are shown in the Standard Details. Each post-tensioning duct contains three ½" diameter strands which produce a total post-tensioning force per duct of 86.7 kips.

Prestressed box girders are subject to high chloride ion exposure because of longitudinal cracking that sometimes occurs between the boxes or from drainage on the fascia girders when an open steel railing system is used. To reduce permeability the concrete mix is required to contain fly ash as stated in 503.2.2 of the Standard Specifications.

When these sections are in contact with water for 5-year flood events or less, the sections must be cast solid for long term durability. When these sections are in contact with water for the 100-year flood event or less, any voids in the section must be cast with a non-water-absorbing material.

Table 19.3-3 provides approximate span limitations for prestressed box girder sections. It also gives the section properties associated with these members. Criteria for developing these tables are shown below Table 19.3-3.

19.3.9.1 Available Prestressed Box Girder Sections and Maximum Span Lengths

Precasters have forms available to make six prestressed girder box sections ranging in depth from 12" to 42". Each section can be made in widths of 36" and 48", but 48" is more efficient and is the preferred width. Typical box section information is shown in the Standard Details.

Table 19.3-3 shows available section depths, section properties, and maximum span length. All sections have voids except the 12" deep section.

	Section No.	Section Depth (inches)	Section Area, A, (in^2)	Moment of Inertia, I, (in^4)	Section Modulus, (in^3) S_{Top}	Section Modulus, (in^3) S_{Bottom}	Torsional Inertia, J, (in^4)	Max. Span (ft)
3'-0" Section Width	1	12	422	5,101	848	852	15,955	24
	2	17	452	14,047	1,648	1,657	23,797	40
	3	21	492	25,240	2,398	2,410	39,928	49
	4	27	565	50,141	3,706	3,722	68,925	58
	5	33	625	85,010	5,142	5,162	102,251	64
	6	42	715	158,749	7,546	7,573	158,033	77
4'-0" Section Width	1	12	566	6,829	1,136	1,140	22,600	25
	2	17	584	18,744	2,201	2,210	38,427	39
	3	21	624	33,501	3,184	3,197	65,395	49
	4	27	697	65,728	4,860	4,877	114,924	59
	5	33	757	110,299	6,674	6,696	173,031	68
	6	42	847	203,046	9,655	9,683	272,267	80

Table 19.3-3
Prestressed Box Girder Section Properties and Maximum Span Length

Table based on:

- HL93 loading and AASHTO LRFD Bridge Design Specifications
- Simple span
- f'_c = 5 ksi and f'_{ci} = 4.25 ksi
- 0.5” dia. or 0.6” dia., low relaxation prestressing strands at $0.75f'_s$
- f'_s = 270.0 ksi
- 6” min. concrete deck (which doesn’t contribute to stiffness of section)
- Single slope parapet 42SS weight distributed evenly to all girder sections
- 30° skew used to compute diaphragm weight
- 2 ¾” of grout between sections
- Post-tensioning diaphragms located as stated in the Standard Details
- 30’-0” minimum clear bridge width (eleven 3’-0” sections, eight 4’-0” sections)

19.3.9.2 Decks and Overlays

There are three types of systems.

1. Reinforced Concrete Deck (design non-composite, detail composite)
2. Concrete Overlay, Grade E or C (non-composite)
3. Asphaltic Overlay with Waterproofing Membrane (not allowed)

19.3.9.3 Grout Between Prestressed Box Girders

These sections are set 1” apart with a ±¼” tolerance. The space between sections is filled with a grout mix prior to post-tensioning the sections transversely. Post-tensioning is not allowed until the grout has cured for at least 48 hours and has attained a compressive strength of 3000psi.

19.4 Field Adjustments of Pretensioning Force

When strands are tensioned in open or unheated areas during cold weather they are subject to loss due to change in temperature. This loss can be compensated for by noting the change in temperature of the strands between tensioning and initial set of the concrete. For purposes of uniformity the strand temperature at initial concrete set is taken as 80°F.

Minor changes in temperature have negligible effects on the prestress force, therefore only at strand temperatures of 50°F and lower are increases in the tensioning force made.

Since plan prestress forces are based on 75% of the ultimate for low relaxation strands it is necessary to utilize the AASHTO allowable of temporarily overstressing up to 80% to provide for the losses associated with fabrication.

The following example outlines these losses and shows the elongation computations which are used in conjunction with jack pressure gages to control the tensioning of the strands.

Computation for Field Adjustment of Prestress Force

Known:

22 - 1/2", 7 wire low relaxation strands, A_{ps} = 0.1531 in^2

P_{pj} = 710.2 kips (jacking force from plan)

T_1 = 40°F (air temperature at strand tensioning)

T_2 = 80°F (concrete temperature at initial set)

L = 300' = 3,600" (distance from anchorage to reference point)

L_1 = 240' = 2,880" (length of cast segment)

E_p = 29,000 ksi (of prestressing tendons, sample tested from each spool)

C = 0.0000065 (coefficient of thermal expansion for steel, per degree F.)

COMPUTE:

jacking force per strand = P_{pj} = 710.2/22 = 32.3 kips

DL_1 = PL/AE = 32.3 x 3600/(0.1531 x 29,000) = 26.1"

Initial Load of 1.5 Kips to set the strands

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E24-1 2-Span Continous Steel Plate Girder Bridge - LRFD

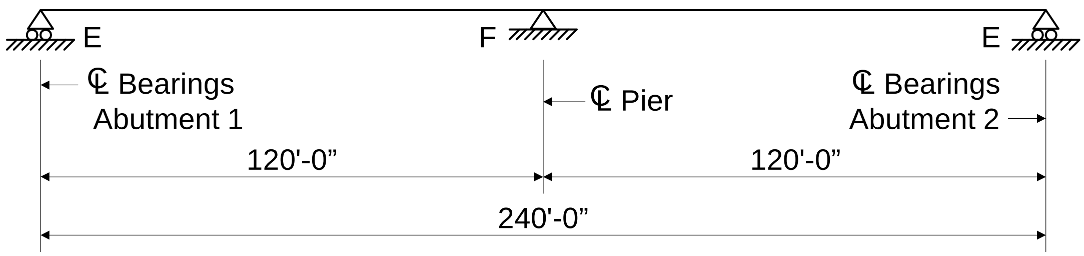
This example shows design calculations conforming to the *AASHTO LRFD Seventh Edition -2014* as supplemented by the *WisDOT Bridge Manual* . Sample design calculations are shown for the following steel superstructure regions or components:

- Interior girder design at the controlling positive moment region
- Interior girder design at the controlling negative moment region
- Transverse stiffener design
- Shear connector design
- Bearing stiffener design

E24-1.1 Obtain Design Criteria

The first design step for a steel girder is to choose the correct design criteria. [24.6.1]

The steel girder design criteria are obtained from Figure E24-1.1-1 through Figure E24-1.1-3 (shown below), and from the referenced articles and tables in the *AASHTO LRFD Bridge Design Specifications, Seventh Edition*. An interior plate girder will be designed for an HL-93 live load for this example. The girder will be designed to be composite throughout. (Note: Figure 5.2-1 contains recommended economical span lengths for steel girders.)



Legend:
E = Expansion Bearings
F = Fixed Bearings

Figure E24-1.1-1
Span Configuration

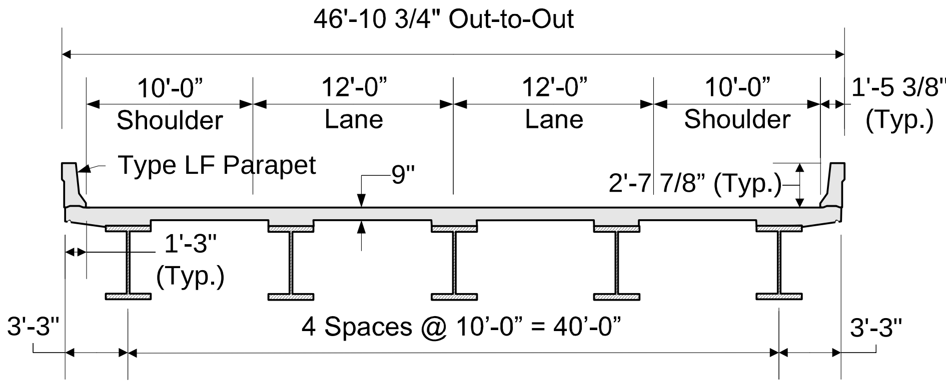


Figure E24-1.1-2
Superstructure Cross Section

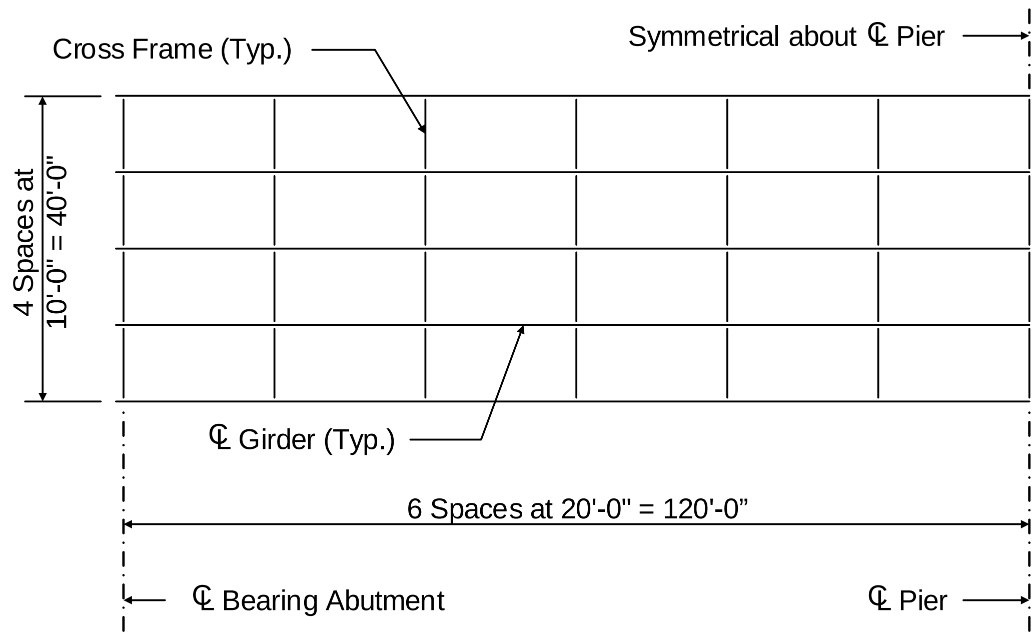


Figure E24-1.1-3
Framing Plan

Design criteria:

$N_{spans} := 2$		Number of spans
$L := 120$	ft	span length
$Skew := 0$	deg	skew angle
$N_b := 5$		number of girders
$S := 10.0$	ft	girder spacing
$S_{overhang} := 3.25$	ft	deck overhang (Per Chapter 17.6.2, WisDOT practice is to limit the overhang to 3'-7", however, economical overhang range is 0.28S - 0.35S based on parapet weight.)
$L_b := 240$	in	cross-frame spacing **LRFD [6.7.4]**
$F_{yw} := 50$	ksi	web yield strength **LRFD [Table 6.4.1-1]**
$F_{yf} := 50$	ksi	flange yield strength **LRFD [Table 6.4.1-1]**
$f'_c := 4.0$	ksi	concrete 28-day compressive strength **LRFD [5.4.2.1 & Table C5.4.2.1-1]**
$f_y := 60$	ksi	reinforcement strength **LRFD [5.4.3 & 6.10.1.7]**

$E_s := 29000$	ksi	modulus of elasticity **LRFD [6.4.1]**
$t_{deck} := 9.0$	in	total deck thickness
$t_s := 8.5$	in	effective deck thickness
$t_{overhang} := 9.5$	in	total overhang thickness
$t_{effoverhang} := 9.0$	in	effective overhang thickness
$W_s := 0.490$	kcf	steel density **LRFD [Table 3.5.1-1]**
$w_c := 0.150$	kcf	concrete density **LRFD [Table 3.5.1-1 & C3.5.1]**
$DL_{misc} := 0.030$	kip/ft	additional miscellaneous dead load (per girder) (Chapter 17.2.4.1)
$W_{par} := 0.464$	kip/ft	parapet weight (each) (Type 32SS)
$W_{fws} := 0.020$	ksf	future wearing surface (Chapter 17.2.4.1)
$w_{deck} := 46.50$	ft	deck width
$w_{roadway} := 44.0$	ft	roadway width
$d_{haunch} := 3.75$	in	haunch depth (from top of web for design) (for construction, the haunch is measured from the top of the top flange)

$ADTT_{SL} := 3000$ Average Daily Truck Traffic (Single-Lane)

Design factors from AASHTO LRFD Bridge Design Specifications:

Load factors,γ, **LRFD [Table 3.4.1-1 & Table 3.4.1-2]**:

Load Combinations and Load Factors							
Limit State	**Load Factors**						
	DC	**DW**	**LL**	**IM**	**WS**	**WL**	**EQ**
Strength I	1.25	1.50	1.75	1.75	-	-	-
Service II	1.00	1.00	1.30	1.30	-	-	-
Fatigue I	-	-	1.50	1.50	-	-	-

Table E24-1.1-1
Load Combinations and Load Factors

The abbreviations used in Table E24-1.1-1 are as defined in **LRFD [3.3.2]**.

The extreme event limit state (including earthquake load) is generally not considered for a steel girder design.

Resistance factors,ϕ, **LRFD [6.5.4.2]**:

Resistance Factors	
Type of Resistance	**Resistance Factor**
For flexure	1.00
For shear	1.00
For axial compression	0.90

Table E24-1.1-2
Resistance Factors

Dynamic load allowance **LRFD [Table 3.6.2.1-1]**:

Dynamic Load Allowance	
Limit State	**Dynamic Load Allowance, IM**
Fatigue and Fracture Limit State	15%
All Other Limit States	33%

Table E24-1.1-3
Dynamic Load Allowance

E24-1.2 Select Trial Girder Section

Before the dead load effects can be computed, a trial girder section must be selected. [24.6.2] This trial girder section is selected based on previous experience and based on preliminary design. For this design example, the trial girder section presented in Figure E24-1.2-1 will be used. Based on this trial girder section, section properties and dead load effects will be computed. Then specification checks will be performed to determine if the trial girder section successfully resists the applied loads. If the trial girder section does not pass all specification checks or if the girder optimization is not acceptable, then a new trial girder section must be selected and the design process must be repeated.

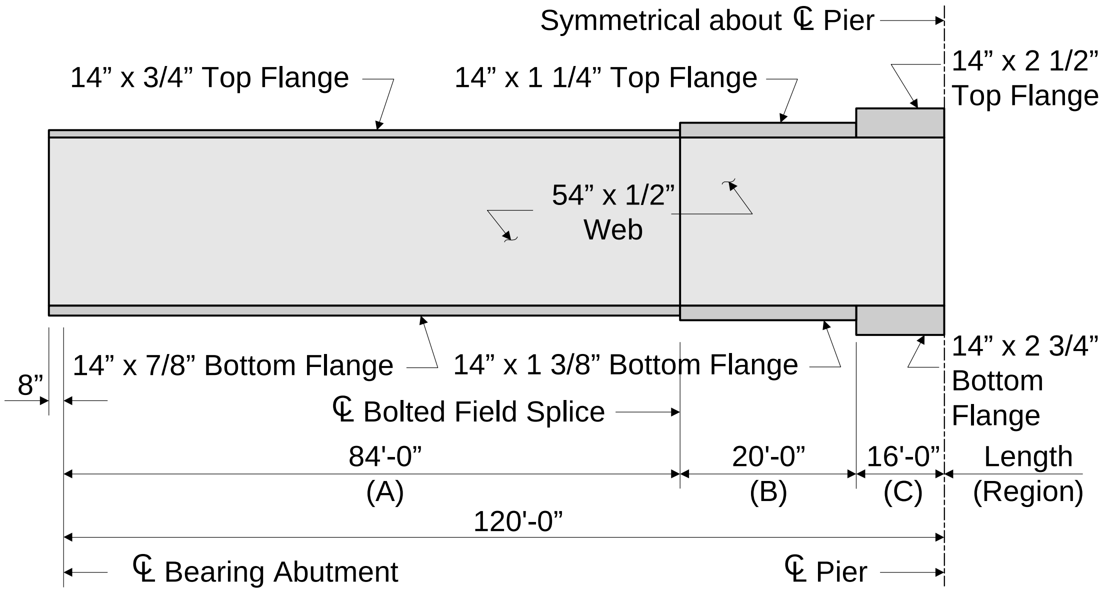


Figure E24-1.2-1
Plate Girder Elevation

The AASHTO/NSBA Steel Bridge Collaboration Document "Guidelines for Design for Constructibility" recommends a 3/4" minimum flange thickness. Wisconsin requires a 3/4" minimum flange thickness.

E24-1.3 Compute Section Properties

Since the superstructure is composite, several sets of section properties must be computed **LRFD [6.10.1.1]**. The initial dead loads (or the noncomposite dead loads) are applied to the girder-only section. For permanent loads assumed to be applied to the long-term composite section, the long-term modular ratio of 3n is used to transform the concrete deck area **LRFD [6.10.1.1.1b]**. For transient loads assumed applied to the short-term composite section, the short-term modular ratio of n is used to transform the concrete deck area.

For girders with shear connectors provided throughout their entire length and with slab reinforcement satisfying the provisions of **LRFD [6.10.1.7]**, stresses due to loads applied to the composite section for the fatigue limit state may be computed using the short-term composite section assuming the concrete slab to be fully effective for both positive and negative flexure **LRFD [6.6.1.2.1 & 6.10.5.1]**.

For girders with shear connectors provided throughout their entire length that also satisfy the provisions of **LRFD [6.10.1.7]**, flexural stresses caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate, assuming the concrete deck is effective for both positive and negative flexure **LRFD [6.10.4.2.1]**.

In general, both the exterior and interior girders must be considered, and the controlling design is used for all girders, both interior and exterior. However, design computations for the interior girder only are presented in this example.

The modular ratio, n, is computed as follows:

$$n := \frac{E_s}{E_c}$$

Where:

E_s	= Modulus of elasticity of steel (ksi)
E_c	= Modulus of elasticity of concrete (ksi)

$E_s = 29000$ ksi **LRFD [6.4.1]**

$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ **LRFD [5.4.2.4]**

Where:

K_1	= Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction. For WisDOT, K_1 = 1.0.
w_c	= Unit weight of concrete (kcf)
f'_c	= Specified compressive strength of concrete (ksi)

$w_c = 0.150$ kcf **LRFD [Table 3.5.1-1 & C3.5.1]**

$f'_c = 4.0$ ksi **LRFD [Table 5.4.2.1-1 & 5.4.2.1]**

$K_1 := 1$ **LRFD [5.4.2.4]**

$E_c := 33000 \cdot K_1 \cdot \left(w_c^{1.5}\right) \cdot \sqrt{f'_c}$ $\boxed{E_c = 3834}$ ksi

$n := \frac{E_s}{E_c}$ $\boxed{n = 7.6}$ **LRFD [6.10.1.1.1b]**

Therefore, use: $n := 8$

The effective flange width is computed as follows (Chapter 17.2.11):

For interior beams, the effective flange width is taken as the average spacing of adjacent beams:

$W_{effflange} := S$ $\boxed{W_{effflange} = 10.00}$ ft

or

$\boxed{W_{effflange} \cdot 12 = 120.00}$ in

Based on Table 17.5-3 of Chapter 17 for a 9" deck and 10'-0" girder spacing, the top mat longitudinal continuity reinforcement bar size and spacing is #6 bars at 7.5" spacing. The area of the top mat longitudinal continuity deck reinforcing steel in the negative moment region is computed below for the effective flange width. For the section properties in Table E24-1.3-3, the location of the centroid of the top mat longitudinal reinforcement is conservatively taken as one-half the structural slab thickness or 8.5" / 2 = 4.25".

$$A_{deckreinf} := 1 \times 0.44 \cdot \frac{W_{effflange} \cdot 12}{7.5} \qquad \boxed{A_{deckreinf} = 7.04} \quad in^2$$

For this design example, the slab haunch is 3.75 inches throughout the length of the bridge. That is, the bottom of the slab is located 3.75 inches above the top of the web (for construction, it is measured from the top of the top flange). For this design example, this distance is used in computing the location of the centroid of the slab. However, the area of the haunch is conservatively not considered in the section properties for this example.

Based on the trial plate sizes shown in Figure E24-1.2-1, the noncomposite and composite section properties for Region A, B, and C are computed as shown in the following tables **LRFD [6.6.1.2.1, 6.10.5.1, 6.10.4.2.1]**. The distance to the centroid is measured from the bottom of the girder.

Region A Section Properties (0 - 84 Feet)						
Section	**Area, A (inches^2)**	**Centroid, d (inches)**	**A*d (inches^3)**	**I_o (inches^4)**	**A*y^2 (inches^4)**	**I_{total} (inches^4)**
Girder only:						
Top flange	10.500	55.250	580.1	0.5	8441.1	8441.6
Web	27.000	27.875	752.6	6561.0	25.8	6586.8
Bottom flange	12.250	0.438	5.4	0.8	8576.1	8576.9
Total	49.750	26.897	1338.1	6562.3	17043.0	23605.3
Composite (3n):						
Girder	49.750	26.897	1338.1	23605.3	13668.5	37273.7
Slab	42.500	62.875	2672.2	255.9	16000.2	16256.0
Total	92.250	43.472	4010.3	23861.1	29668.6	53529.8
Composite (n):						
Girder	49.750	26.897	1338.1	23605.3	33321.4	56926.6
Slab	127.500	62.875	8016.6	767.7	13001.9	13769.5
Total	177.250	52.777	9354.7	24372.9	46323.2	70696.2
Section	**y_{botgdr} (inches)**	**y_{topgdr} (inches)**	**$y_{topslab}$ (inches)**	**S_{botgdr} (inches^3)**	**S_{topgdr} (inches^3)**	**$S_{topslab}$ (inches^3)**
Girder only	26.897	28.728	---	877.6	821.7	---
Composite (3n)	43.472	12.153	23.653	1231.4	4404.7	2263.1
Composite (n)	52.777	2.848	14.348	1339.5	24820.6	4927.1

Table E24-1.3-1
Region A Section Properties

Region B Section Properties (84 - 104 Feet)						
Section	**Area, A (inches^2)**	**Centroid, d (inches)**	**A*d (inches^3)**	**I_o (inches^4)**	**A*y^2 (inches^4)**	**I_{total} (inches^4)**
Girder only:						
Top flange	17.500	56.000	980.0	2.3	14117.0	14119.3
Web	27.000	28.375	766.1	6561.0	16.3	6577.3
Bottom flange	19.250	0.688	13.2	3.0	13940.2	13943.2
Total	63.750	27.598	1759.4	6566.3	28073.5	34639.8
Composite (3n):						
Girder	63.750	27.598	1759.4	34639.8	13056.1	47695.9
Slab	42.500	63.375	2693.4	255.9	19584.1	19840.0
Total	106.250	41.909	4452.8	34895.7	32640.2	67535.9
Composite (n):						
Girder	63.750	27.598	1759.4	34639.8	36266.9	70906.7
Slab	127.500	63.375	8080.3	767.7	18133.5	18901.1
Total	191.250	51.449	9839.7	35407.4	54400.4	89807.8
Section	**y_{botgdr} (inches)**	**y_{topgdr} (inches)**	**$y_{topslab}$ (inches)**	**S_{botgdr} (inches^3)**	**S_{topgdr} (inches^3)**	**$S_{topslab}$ (inches^3)**
Girder only	27.598	29.027	---	1255.2	1193.4	---
Composite (3n)	41.909	14.716	25.716	1611.5	4589.2	2626.2
Composite (n)	51.449	5.176	16.176	1745.6	17351.7	5552.0

Table E24-1.3-2
Region B Section Properties

For the strength limit state, since the deck concrete is in tension in the negative moment region, the deck reinforcing steel contributes to the composite section properties and the deck concrete does not.

As previously explained, for this design example, the concrete slab will be assumed to be fully effective for both positive and negative flexure for service and fatigue limit states.

Region C Section Properties (104 - 120 Feet)						
Section	Area, A (inches^2)	Centroid, d (inches)	A*d (inches^3)	I_o (inches^4)	A*y^2 (inches^4)	I_{total} (inches^4)
Girder only:						
Top flange	35.000	58.000	2030.0	18.2	30009.7	30027.9
Web	27.000	29.750	803.3	6561.0	28.7	6589.7
Bottom flange	38.500	1.375	52.9	24.3	28784.7	28809.0
Total	100.500	28.718	2886.2	6603.5	58823.1	65426.6
Composite (deck concrete using 3n):						
Girder	100.500	28.718	2886.2	65426.6	11525.0	76951.6
Slab	42.500	64.750	2751.9	255.9	27253.3	27509.2
Total	143.000	39.427	5638.1	65682.5	38778.3	104460.8
Composite (deck concrete using n):						
Girder	100.500	28.718	2886.2	65426.6	40802.5	106229.1
Slab	127.500	64.750	8255.6	767.7	32162.0	32929.6
Total	228.000	48.868	11141.8	66194.3	72964.4	139158.7
Composite (top longitudinal deck reinforcement only):						
Girder	100.500	28.718	2886.2	65426.6	559.2	65985.8
Deck reinf.	7.040	64.750	455.8	0.0	7982.4	7982.4
Total	107.540	31.077	3342.0	65426.6	8541.6	73968.2
Section	y_{botgdr} (inches)	y_{topgdr} (inches)	y_{deck} (inches)	S_{botgdr} (inches^3)	S_{topgdr} (inches^3)	S_{deck} (inches^3)
Girder only	28.718	30.532	---	2278.2	2142.9	---
Composite (3n)	39.427	19.823	29.573	2649.5	5269.7	3532.3
Composite (n)	48.868	10.382	20.132	2847.7	13403.3	6912.2
Composite (rebar	31.077	28.173	33.673	2380.2	2625.5	2196.7

Table E24-1.3-3
Region C Section Properties

The section properties used to compute the unfactored dead and live load moments and shears for each girder region are given in the following table in accordance with the requirements of **LRFD [6.10.1.5]**.

Girder Region (ft)	Moment of Inertia Used (in^4)		
	Beam Self Weight, Misc Dead Loads, Concrete Deck & Haunch (Noncomposite)	Wisconsin Barrier, Future Wearing Surface (Composite)	HI-93 Live Load (Composite)
Region A (0-84)	23605.3	53529.8	70696.2
Region B (84-104)	34639.8	67535.9	89807.8
Region C (104-120)	65426.6	104460.8	139158.7

Table E24-1.3-4
Section Properties Used to Generate Design Moments and Shears

E24-1.4 Compute Dead Load Effects

The girder must be designed to resist the dead load effects, as well as the other load effects. The dead load components consist of some dead loads that are resisted by the noncomposite section, as well as other dead loads that are resisted by the composite section. In addition, some dead loads are factored with the DC load factor and other dead loads are factored with the DW load factor. The following table summarizes the various dead load components that must be included in the design of a steel girder.

Dead Load Components		
Resisted by	**Type of Load Factor**	
	DC	**DW**
Noncomposite section	• Steel girder • Concrete deck • Concrete haunch • Stay-in-place deck forms • Miscellaneous dead load (including cross-frames, stiffeners, etc.)	
Composite section	• Concrete parapets	• Future wearing surface & utilities

Table E24-1.4-1
Dead Load Components

For the steel girder, the dead load per unit length varies due to the change in plate sizes. The moments and shears due to the weight of the steel girder can be computed using readily available analysis software. Since the actual plate sizes are entered as input, the moments and shears are computed based on the actual, varying plate sizes.

The dead load per unit length for Regions A, B, and C is calculated as follows:

$A_A = 49.75$ in^2 Region A (0 - 84 feet)(Table E24-1.3-1)

$A_B = 63.75$ in^2 Region B (84 - 104 feet)(Table E24-1.3-2)

$A_C = 100.50$ in^2 Region C (104 - 120 feet)(Table E24-1.3-3)

Weight of Girder per region:

$$DL_A := W_s \cdot \frac{A_A}{12^2} \qquad \boxed{DL_A = 0.169} \quad \text{klf}$$

$$DL_B := W_s \cdot \frac{A_B}{12^2} \qquad \boxed{DL_B = 0.217} \quad \text{klf}$$

$DL_C := W_S \cdot \frac{A_C}{12^2}$ $\boxed{DL_C = 0.342}$ klf

For the concrete deck, the dead load per unit length for an interior girder is computed as follows:

$w_c = 0.150$ kcf

$S = 10.00$ ft

$t_{deck} = 9.00$ in

$DL_{deck} := w_c \cdot S \cdot \frac{t_{deck}}{12}$ $\boxed{DL_{deck} = 1.125}$ kip/ft

For the concrete haunch, the dead load per unit length varies due to the change in top flange plate sizes. The moments and shears due to the concrete haunch are computed based on the actual, varying haunch thickness.

The haunch dead load per unit length for Region A, B, and C is calculated as follows:

$width_{flange} := 14$ in Top flange width is consistent in all three regions.

$t_{flangeA} := 0.75$ in Top flange thickness in Region A

$t_{flangeB} := 1.25$ in Top flange thickness in Region B

$t_{flangeC} := 2.5$ in Top flange thickness in Region C

$d_{haunch} = 3.75$ in Distance from top of web to bottom of deck as detailed in E24-1.1

$d_{hA} := d_{haunch} - t_{flangeA}$ $\boxed{d_{hA} = 3.00}$ in

$d_{hB} := d_{haunch} - t_{flangeB}$ $\boxed{d_{hB} = 2.50}$ in

$d_{hC} := d_{haunch} - t_{flangeC}$ $\boxed{d_{hC} = 1.25}$ in

$w_c = 0.150$ kcf

$DL_{hA} := \frac{width_{flange} \cdot d_{hA}}{12^2} \cdot w_c$ $\boxed{DL_{hA} = 0.044}$ klf

$DL_{hB} := \frac{width_{flange} \cdot d_{hB}}{12^2} \cdot w_c$ $\boxed{DL_{hB} = 0.036}$ klf

$$DL_{hC} := \frac{width_{flange} \cdot d_{hC}}{12^2} \cdot w_c \qquad \boxed{DL_{hC} = 0.018} \quad \text{klf}$$

Total weight of deck and haunch per region:

$$DL_{dhA} := DL_{deck} + DL_{hA} \qquad \boxed{DL_{dhA} = 1.169} \quad \text{klf}$$

$$DL_{dhB} := DL_{deck} + DL_{hB} \qquad \boxed{DL_{dhB} = 1.161} \quad \text{klf}$$

$$DL_{dhC} := DL_{deck} + DL_{hC} \qquad \boxed{DL_{dhC} = 1.143} \quad \text{klf}$$

For the miscellaneous dead load (including cross-frames, stiffeners, and other miscellaneous structural steel), the dead load per unit length is assumed to be as follows:

$DL_{misc} = 0.030$ kip/ft See E24-1.1

For the concrete parapets, the dead load per unit length is computed as follows, assuming that the superimposed dead load of the two parapets is distributed uniformly among all of the girders **LRFD [4.6.2.2.1]**:

$W_{par} = 0.464$ kip/ft (Type LF)

$N_b = 5$

$$DL_{par} := \frac{W_{par} \cdot 2}{N_b} \qquad \boxed{DL_{par} = 0.186} \quad \text{kip/ft}$$

For the future wearing surface, the dead load per unit length is computed as follows, assuming that the superimposed dead load of the future wearing surface is distributed uniformly among all of the girders **LRFD [4.6.2.2.1]**:

$W_{fws} = 0.020$ ksf

$w_{roadway} = 44.0$ ft

$N_b = 5$

$$DL_{fws} := \frac{W_{fws} \cdot w_{roadway}}{N_b} \qquad \boxed{DL_{fws} = 0.176} \quad \text{kip/ft}$$

Since the plate girder and its section properties are not uniform over the entire length of the bridge, an analysis software was used to compute the dead load moments and shears.

The following two tables present the unfactored dead load moments and shears, as computed by an analysis computer program. Since the bridge is symmetrical, the moments and shears in Span 2 are symmetrical to those in Span 1.

Dead Load Moments (Kip-feet)											
Dead Load Component	**Location in Span 1**										
	0.0L	**0.1L**	**0.2L**	**0.3L**	**0.4L**	**0.5L**	**0.6L**	**0.7L**	**0.8L**	**0.9L**	**1.0L**
Steel girder	0.0	71.7	119.1	142.1	140.7	114.9	64.8	-9.8	-112.1	-246.9	-430.4
Concrete deck & haunches	0.0	487.6	808.0	961.3	947.3	766.2	417.9	-97.6	-780.3	-1630.2	-2647.3
Other dead loads acting on girder alone	0.0	12.9	21.5	25.7	25.7	21.3	12.6	-0.4	-17.8	-39.5	-65.4
Concrete parapets	0.0	80.0	133.1	159.5	159.1	131.9	78.0	-2.8	-110.3	-244.6	-405.7
Future wearing surface	0.0	75.7	126.0	150.9	150.6	124.8	73.8	-2.6	-104.4	-231.5	-383.9

Table 24E1.4-2
Dead Load Moments

Dead Load Shears (Kips)											
Dead Load Component	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Steel girder	7.0	5.0	2.9	0.9	-1.1	-3.2	-5.2	-7.2	-9.8	-13.1	-17.5
Concrete deck & haunches	47.6	33.7	19.7	5.8	-8.1	-22.1	-36.0	-49.9	-63.9	-77.8	-91.7
Other dead loads acting on girder alone	1.3	0.9	0.5	0.2	-0.2	-0.5	-0.9	-1.3	-1.6	-2.0	-2.3
Concrete parapets	7.8	5.5	3.3	1.1	-1.1	-3.4	-5.6	-7.8	-10.1	-12.3	-14.5
Future wearing surface	7.4	5.2	3.1	1.0	-1.1	-3.2	-5.3	-7.4	-9.5	-11.6	-13.8

Table 24E1.4-3
Dead Load Shears

E24-1.5 Compute Live Load Effects

The girder must also be designed to resist the live load effects **LRFD [3.6.1.2]**. The live load consists of an HL-93 loading. Similar to the dead load, the live load moments and shears for an HL-93 loading were obtained from an analysis computer program.

Based on Table E24-1.1-3, for all limit states other than fatigue and fracture, the dynamic load allowance, IM, is as follows **LRFD [3.6.2.1]**:

$$IM := 0.33$$

The live load distribution factors for moment for an interior girder are computed as follows **LRFD [4.6.2.2.2]**:

First, the longitudinal stiffness parameter, K_g, must be computed **LRFD [4.6.2.2.1]**:

$$K_g := n \cdot \left(I + A \cdot e_g^{\,2}\right)$$

Where:

I = Moment of inertia of beam (in^4)

A = Area of stringer, beam, or girder (in^2)

e_g = Distance between the centers of gravity of the basic beam and deck (in)

Longitudinal Stiffness Parameter, K_g				
	Region A (Pos. Mom.)	**Region B (Intermediate)**	**Region C (At Pier)**	**Weighted Average ***
Length (Feet)	84	20	16	
n	8	8	8	
I (Inches^4)	23,605.3	34,639.8	65,426.6	
A (Inches^2)	49.750	63.750	100.500	
e_g (Inches)	35.978	35.777	36.032	
K_g (Inches^4)	704,020	929,915	1,567,250	856,767

Table E24-1.5-1
Longitudinal Stiffness Parameter

After the longitudinal stiffness parameter is computed, **LRFD [Table 4.6.2.2.1-1]** is used to find the letter corresponding with the superstructure cross section. The letter corresponding with the superstructure cross section in this design example is "a."

If the superstructure cross section does not correspond with any of the cross sections illustrated in **LRFD [Table 4.6.2.2.1-1]**, then the bridge should be analyzed as presented in **LRFD [4.6.3]**.

Based on cross section "a", **LRFD [Table 4.6.2.2.2b-1 & Table 4.6.2.2.3a-1]** are used to compute the distribution factors for moment and shear, respectively.

Check the range of applicability as follows **LRFD [Table 4.6.2.2.2b-1]**:

$\boxed{3.5 \le S \le 16.0}$

Where:

S = Spacing of beams or webs (ft)

$S = 10.00$ ft OK

$\boxed{4.5 \le t_s \le 12.0}$

Where:

t_s = Depth of concrete slab (in)

$t_s = 8.5$ in OK

$\boxed{20 \le L \le 240}$

Where:

L = Span of beam (ft)

$L := 120$ ft OK

$\boxed{N_b \ge 4}$

Where:

N_b = Number of beams, stringers, or girders

$N_b = 5.00$ OK

$\boxed{10000 \le K_g \le 7000000}$

$K_g := 856767$ in^4 OK

For one design lane loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [4.6.2.2.2b-1]**:

$$g_{int\_moment\_1} := 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0L \cdot t_s^{\,3}}\right)^{0.1}$$

$\boxed{g_{int\_moment\_1} = 0.473}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for moment in interior beams is as follows **LRFD [Table 4.6.2.2.2b-1]**:

$$g_{int\_moment\_2} := 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0 \cdot L \cdot t_s^{3}}\right)^{0.1}$$

$\boxed{g_{int\_moment\_2} = 0.700}$ lanes

The live load distribution factors for shear for an interior girder are computed in a similar manner. The range of applicability is similar to that for moment **LRFD [Table 4.6.2.2.3a-1]**.

For one design lane loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{int\_shear\_1} := 0.36 + \frac{S}{25.0}$$

$\boxed{g_{int\_shear\_1} = 0.760}$ lanes

For two or more design lanes loaded, the distribution of live load per lane for shear in interior beams is as follows:

$$g_{int\_shear\_2} := 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$\boxed{g_{int\_shear\_2} = 0.952}$ lanes

Since this bridge has no skew, the skew correction factor does not need to be considered for this design example **LRFD [4.6.2.2.2e & 4.6.2.2.3c]**.

This design example is based on an interior girder. However, for illustrative purposes, the live load distribution factors for an exterior girder are computed below, as follows **LRFD [4.6.2.2.2]**:

The distance, d_e, is defined as the distance between the web centerline of the exterior girder and the interior edge of the curb. For this design example, based on Figure E24-1.1-2:

$d_e := S_{overhang} - 1.25$ ft

Check the range of applicability as follows **LRFD [Table 4.6.2.2.2d-1]**:

$-1.0 \le d_e \le 5.5$

$\boxed{d_e = 2.00}$ ft OK

For one design lane loaded, the distribution of live load per lane for moment in exterior beams is computed using the lever rule, as follows:

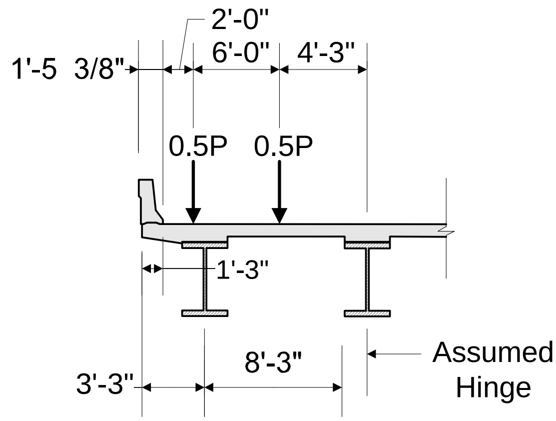


Figure E24-1.5-1
Lever Rule

$$x_1 := S - 6 + (d_e - 2) \qquad x_2 := S + (d_e - 2)$$

$$g_{ext\_moment\_1} := \frac{(0.5)\cdot(x_1) + (0.5)\cdot(x_2)}{S} \qquad \boxed{g_{ext\_moment\_1} = 0.700} \text{ lanes}$$

$$mpf := 1.20$$

$$g_{ext\_moment\_1} := g_{ext\_moment\_1}\cdot mpf \qquad \boxed{g_{ext\_moment\_1} = 0.840}$$

lanes
(for strength limit state)

For two or more design lanes loaded, the distribution of live load per lane for moment in exterior beams is as follows **LRFD [Table 4.6.2.2.2d-1]**:

The correction factor for distribution, e, is computed as follows:

$$e := 0.77 + \frac{d_e}{9.1} \qquad \boxed{e = 0.990}$$

$$g_{ext\_moment\_2} := e\cdot g_{int\_moment\_2} \qquad \boxed{g_{ext\_moment\_2} = 0.693} \text{ lanes}$$

The live load distribution factors for shear for an exterior girder are computed in a similar manner. The range of applicability is similar to that for moment **LRFD [Table 4.6.2.2.3.b-1]**.

For one design lane loaded, the distribution of live load per lane for shear in exterior beams is computed using the lever rule, as illustrated in Figure E24-1.5-1 and as follows:

$$g_{ext\_shear\_1} := \frac{(0.5)\cdot(x_1) + (0.5)\cdot(x_2)}{S} \qquad \boxed{g_{ext\_shear\_1} = 0.700} \text{ lanes}$$

$$g_{ext\_shear\_1} := g_{ext\_shear\_1}\cdot mpf \qquad \boxed{g_{ext\_shear\_1} = 0.840}$$

lanes
(for strength limit state)

For two or more design lanes loaded, the distribution of live load per lane for shear in exterior beams is as follows **LRFD [Table 4.6.2.2.3b-1]**:

$$e := 0.6 + \frac{d_e}{10}$$ $\boxed{e = 0.800}$

$$g_{ext\_shear\_2} := e \cdot g_{int\_shear\_2}$$ $\boxed{g_{ext\_shear\_2} = 0.761}$ lanes

In beam-slab bridge cross-sections with diaphragms or cross-frames, the distribution factor for the exterior beam can not be taken to be less than that which would be obtained by assuming that the cross-section deflects and rotates as a rigid cross-section. **LRFD [C4.6.2.2.2d]** provides one approximate approach to satisfy this requirement. The multiple presence factor provisions of **LRFD [3.6.1.1.2]** must be applied when this equation is used. This is not shown here since an interior girder is being designed.

Since this bridge has no skew, the skew correction factor does not need to be considered for this design example **LRFD [4.6.2.2.2e & 4.6.2.2.3c]**.

The controlling distribution factors for moment and shear for the interior girder are given below.

Interior Girder Distribution Factors		
	Moment DF	**Shear DF**
One Lane	0.473	0.760
Two or More Lanes	0.700	0.952

Table E24-1.5-2
Summary of Interior Girder Distribution Factors

The following table presents the unfactored maximum positive and negative live load moments and shears for HL-93 live loading for interior beams, as computed using an analysis computer program. These values include the controlling live load distribution factor given above for two or more lanes, and they also include dynamic load allowance. Since the bridge is symmetrical, the moments and shears in Span 2 are symmetrical to those in Span 1.

Live Load Effects (for Interior Beams)											
Live Load Effect	Location in Span 1										
	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Maximum positive moment (K-ft)	0.0	858.4	1449.2	1792.8	1923.5	1858.2	1616.5	1207.4	684.9	264.2	0.0
Maximum negative moment (K-ft)	0.0	-143.2	-286.4	-429.6	-572.8	-716.0	-859.2	-1002.4	-1165.9	-1724.3	-2571.5
Maximum positive shear (kips)	112.8	94.1	76.7	60.8	46.5	34.0	23.3	14.5	7.6	3.0	0.0
Maximum negative shear (kips)	-16.2	-16.7	-22.8	-36.3	-50.8	-65.6	-80.4	-94.9	-108.8	-122.1	-134.9

Table 24E1.5-2
Live Load Effects

The design live load values for HL-93 loading, as presented in the previous table, are computed based on the product of the live load effect per lane and live load distribution factor. These values also include the effects of dynamic load allowance. However, it is important to note that the dynamic load allowance is applied only to the design truck or tandem. The dynamic load allowance is not applied to pedestrian loads or to the design lane load **LRFD [3.6.1, 3.6.2, 4.6.2.2]**.

E24-1.6 Combine Load Effects

After the load factors and load combinations have been established (see E24-1.1), the section properties have been computed (see E24-1.3), and all of the load effects have been computed (see E24-1.4 and E24-1.5), the force effects must be combined for each of the applicable limit states.

For this design example, η equals 1.00 **LFRD[1.3]**. (For more detailed information about η, refer to E24-1.1.)

The maximum positive moment (located at 0.4L) for the Strength I Limit State is computed as follows **LRFD [3.4.1]**:

$LF_{DC} := 1.25$

$M_{DC} := 140.7 + 947.3 + 25.7 + 159.1$ $\boxed{M_{DC} = 1272.8}$ kip-ft

$LF_{DW} := 1.50$

$M_{DW} := 150.6$ kip-ft

$LF_{LL} := 1.75$

$M_{LL} := 1923.5$

$M_{total} := LF_{DC} \cdot M_{DC} + LF_{DW} \cdot M_{DW} + LF_{LL} \cdot M_{LL}$ $\boxed{M_{total} = 5183.0}$ kip-ft

Similarly, the maximum stress in the top of the girder due to positive moment (located at 0.4L) for the Strength I Limit State is computed as follows:

Noncomposite dead load:

$M_{noncompDL} := 140.7 + 947.3 + 25.7$ $\boxed{M_{noncompDL} = 1113.70}$ kip-ft

$S_{topgdr} := 821.7$ in^3

$f_{noncompDL} := \dfrac{-M_{noncompDL} \cdot (12)}{S_{topgdr}}$ $\boxed{f_{noncompDL} = -16.26}$ ksi

Parapet dead load (composite):

$M_{parapet} := 159.1$ kip-ft

$S_{topgdr} := 4404.7$ in^3

$$f_{parapet} := \frac{-M_{parapet} \cdot (12)}{S_{topgdr}} \qquad \boxed{f_{parapet} = -0.43} \quad \text{ksi}$$

Future wearing surface dead load (composite):

$M_{fws} := 150.6$ kip-ft

$S_{topgdr} := 4404.7$ in^3

$$f_{fws} := \frac{-M_{fws} \cdot (12)}{S_{topgdr}} \qquad \boxed{f_{fws} = -0.41} \quad \text{ksi}$$

Live load (HL-93) and dynamic load allowance:

$M_{LL} = 1923.50$ kip-ft

$S_{topgdr} := 24820.6$ in^3

$$f_{LL} := \frac{-M_{LL} \cdot (12)}{S_{topgdr}} \qquad \boxed{f_{LL} = -0.93} \quad \text{ksi}$$

Multiplying the above stresses by their respective load factors and adding the products results in the following combined stress for the Strength I Limit State **LRFD [3.4.1]**:

$$f_{Str} := \left(LF_{DC} \cdot f_{noncompDL}\right) + \left(LF_{DC} \cdot f_{parapet}\right) + \left(LF_{DW} \cdot f_{fws}\right) + \left(LF_{LL} \cdot f_{LL}\right)$$

$$\boxed{f_{Str} = -23.12} \quad \text{ksi}$$

Similarly, all of the combined moments, shears, and flexural stresses can be computed at the controlling locations. A summary of those combined load effects for an interior beam is presented in the following three tables, summarizing the results obtained using the procedures demonstrated in the above computations.

Combined Effects at Location of Maximum Positive Moment				
Summary of Unfactored Values:				
Loading	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**$f_{topslab}$ (ksi)**
Noncomposite DL	1102.0	15.07	-16.09	0.00
Parapet DL	136.9	1.35	-0.41	-0.04
FWS DL	155.4	1.53	-0.47	-0.05
LL - HL-93	1916.6	17.27	-1.18	-0.62
LL - Fatigue Range	871.4	7.85	-0.54	-0.28
Summary of Factored Values:				
Limit State	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**$f_{topslab}$ (ksi)**
Strength I	5135.8	53.03	-23.40	-1.22
Service II	3885.9	40.39	-18.51	-0.91
Fatigue I	653.5	5.89	-0.40	-0.21

Table E24-1.6-1

Combined Effects at Location of Maximum Positive Moment

As shown in the above table, the Strength I Limit State elastic stress in the bottom of the girder exceeds the girder yield stress. However, for this design example, this value is not used because of the local yielding that is permitted to occur at this section at the strength limit state.

Combined Effects at Location of Maximum Negative Moment				
Summary of Unfactored Values (Assuming Concrete Not Effective):				
Loading	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Noncomposite DL	-3073.2	-16.19	17.21	0.00
Parapet DL	-327.5	-1.66	1.52	1.82
FWS DL	-371.9	-1.88	1.73	2.06
LL - HL-93	-2414.2	-12.21	11.23	13.40
Summary of Unfactored Values (Assuming Concrete Effective):				
Loading	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Noncomposite DL	-3073.2	-16.19	17.21	0.00
Parapet DL	-327.5	-1.49	0.79	0.08
FWS DL	-371.9	-1.70	0.90	0.09
LL - HL-93	-2414.2	-10.23	2.39	0.56
LL - Fatigue Range	-481.4	-2.04	0.48	0.11
Summary of Factored Values:				
Limit State	**Moment (K-ft)**	**f_{botgdr} (ksi)**	**f_{topgdr} (ksi)**	**f_{deck} (ksi)**
Strength I *	-9033.6	-46.50	45.66	28.82
Service II **	-6911.1	-32.67	22.01	0.89
Fatigue I **	-361.0	-1.53	0.36	0.08

Legend:

* Strength I Limit State stresses are based on section properties assuming the deck concrete is not effective, and f_{deck} is the stress in the deck reinforcing steel.

** Service II and Fatigue I Limit State stresses are based on section properties assuming the deck concrete is effective, and f_{deck} is the stress in the deck concrete.

Table E24-1.6-2

Combined Effects at Location of Maximum Negative Moment

Combined Effects at Location of Maximum Shear	
Summary of Unfactored Values:	
Loading	**Shear (kips)**
Noncomposite DL	108.9
Parapet DL	12.0
FWS DL	13.7
LL - HL-93	132.0
LL - Fatigue Range	56.5
Summary of Factored Values:	
Limit State	**Shear (kips)**
Strength I	402.7
Service II	306.2
Fatigue I	42.4

Table E24-1.6-3
Combined Effects at Location of Maximum Shear

Envelopes of the factored Strength I moments and shears are presented in the following two figures. Maximum and minimum values are presented. As mentioned previously, all remaining design computations in this example are based on the interior girder. The basic approach illustrated in the subsequent design calculations applies equally to the exterior and interior girders (with some exceptions noted) once the load effects in each girder have been determined.

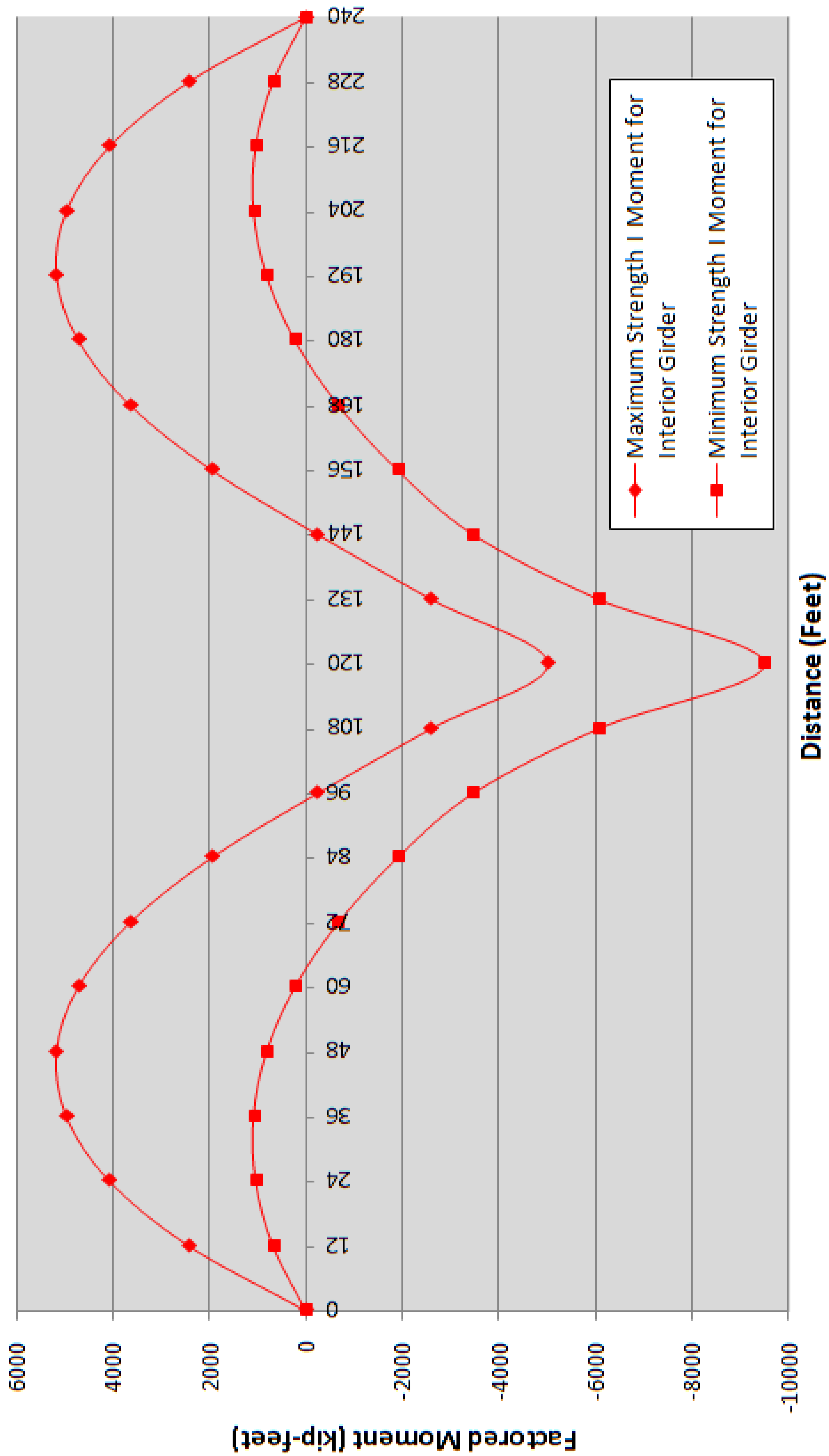


Figure 24E1.6-1

Envelope of Strength I Moments

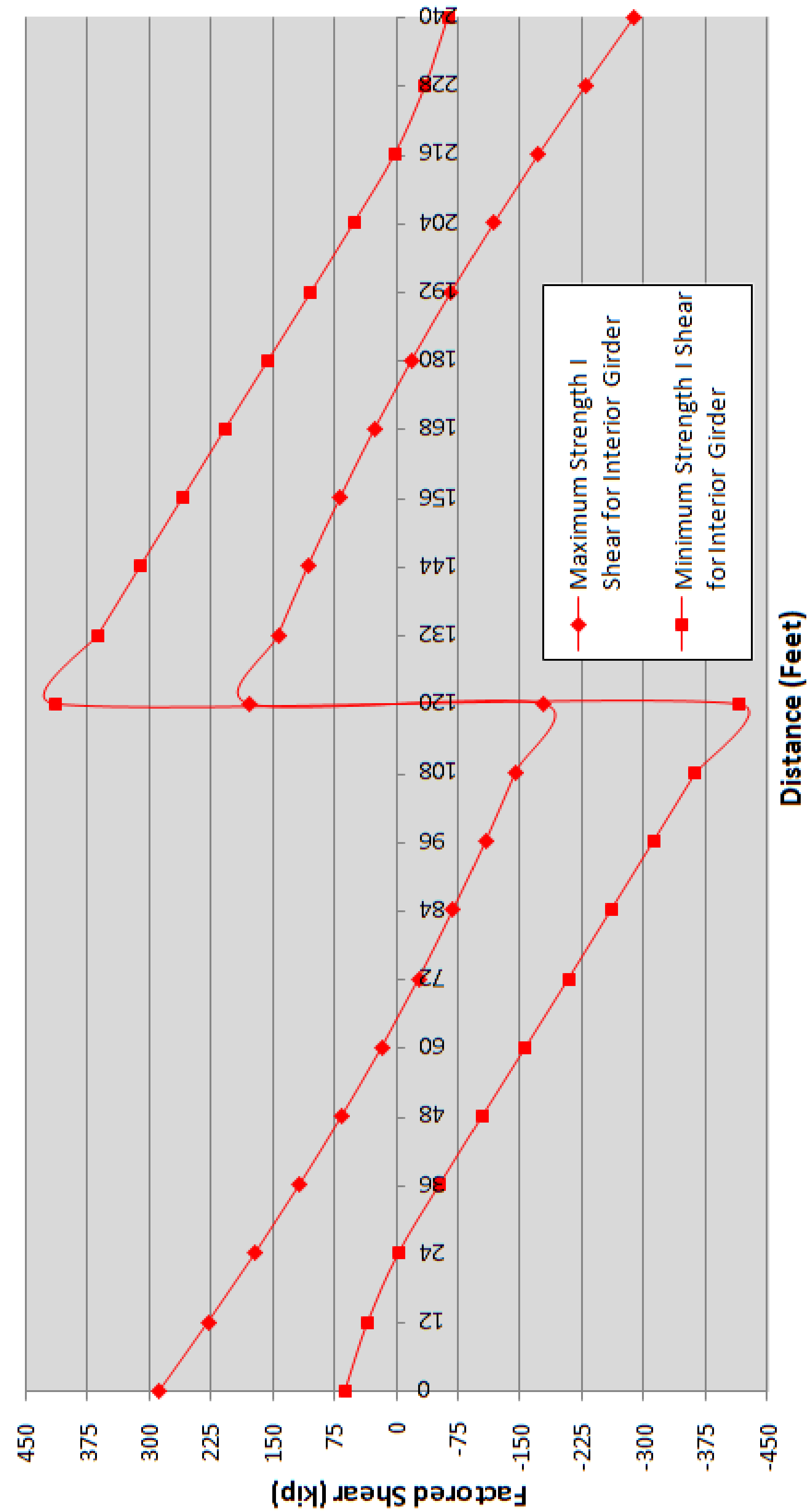


Figure 24E1.6-2
Envelope of Strength I Shears

Two design sections will be checked for illustrative purposes. First, all specification checks will be performed for the location of maximum positive moment, which is at 0.4L in Span 1. Second, all specification checks for these same design steps will be performed for the location of maximum negative moment and maximum shear, which is at the pier.

The following specification checks are for the location of maximum positive moment, which is at 0.4L in Span 1, as shown in Figure E24-1.6-3.

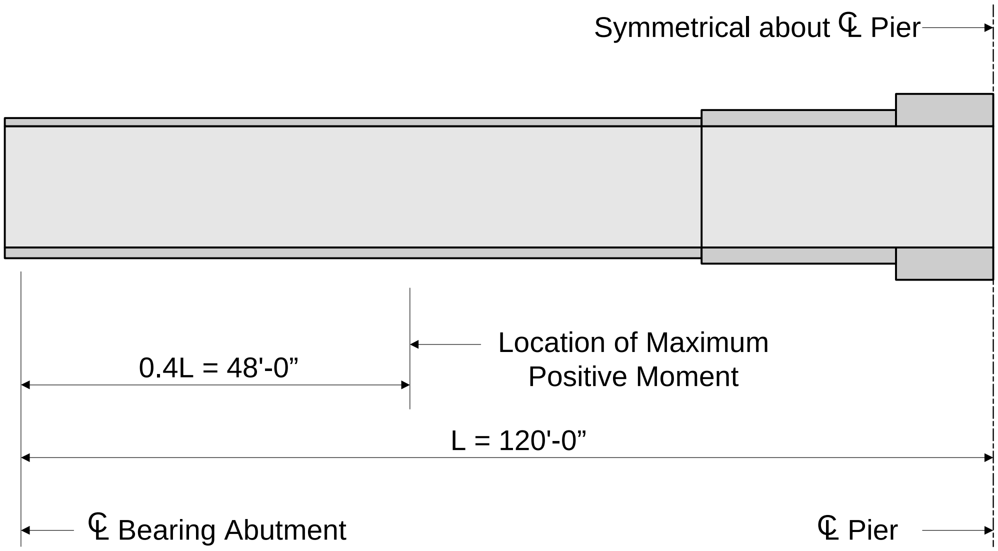


Figure E24-1.6-3
Location of Maximum Positive Moment

E24-1.7 Check Section Proportion Limits - Positive Moment Region

Several checks are required to ensure that the proportions of the trial girder section are within specified limits **LRFD [6.10.2]**.

The first section proportion check relates to the web slenderness **LRFD [6.10.2.1]**. For a section without longitudinal stiffeners, the web must be proportioned such that:

$$\frac{D}{t_w} \le 150$$

Where:

D	= Clear distance between flanges (in)
t_w	= Web thickness (in)

$D := 54$ in

$t_w := 0.50$ in

$$\boxed{\frac{D}{t_w} = 108.00}$$ OK

The second set of section proportion checks relate to the general proportions of the section **LRFD [6.10.2.2]**. The compression and tension flanges must be proportioned such that:

$$\frac{b_f}{2 \cdot t_f} \le 12.0$$

Where:

b_f = Full width of the flange (in)

t_f = Flange thickness (in)

$b_f := 14$ $t_f := 0.75$ $\boxed{\frac{b_f}{2 \cdot t_f} = 9.33}$ OK

$b_f \ge \frac{D}{6}$ $\boxed{\frac{D}{6} = 9.00}$ in OK

$t_f \ge 1.1 \cdot t_w$ $\boxed{1.1 t_w = 0.55}$ in OK

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10$$

Where:

I_{yc} = moment of inertia of the compression flange of a steel section about the vertical axis in the plane of the web (in^4)

I_{yt} = moment of inertia of the tension flange of a steel section about the vertical axis in the plane of the web (in^4)

$I_{yc} := \frac{0.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 171.50}$ in^4

$I_{yt} := \frac{0.875 \cdot 14^3}{12}$ $\boxed{I_{yt} = 200.08}$ in^4

$\boxed{\frac{I_{yc}}{I_{yt}} = 0.857}$ OK

E24-1.8 Compute Plastic Moment Capacity - Positive Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**.

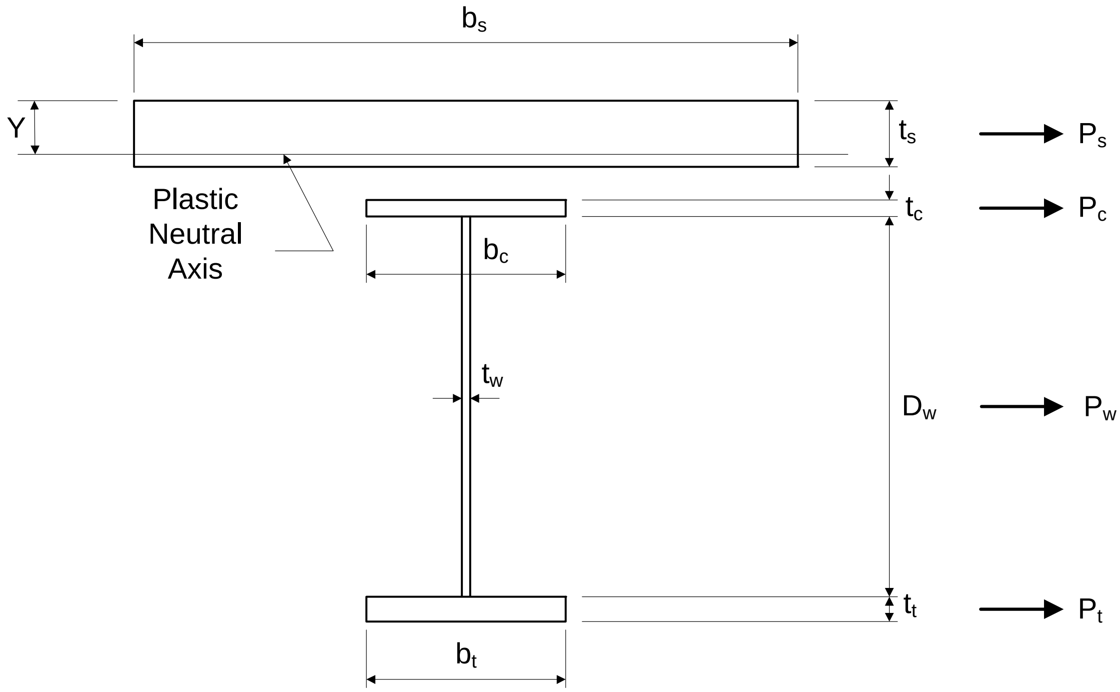


Figure E24-1.8-1
Computation of Plastic Moment Capacity for Positive Bending Sections

For the tension flange:

$P_t := F_{yt} \cdot b_t \cdot t_t$

Where:

F_{yt}	= Specified minimum yield strength of a tension flange (ksi)
b_t	= Full width of the tension flange (in)
t_t	= Thickness of tension flange (in)

$F_{yt} := 50$ ksi

$b_t := 14$ in

$t_t := 0.875$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 613}$ kips

For the web:

$P_w := F_{yw} \cdot D \cdot t_w$

Where:

F_{yw} = Specified minimum yield strength of a web (ksi)

$F_{yw} = 50$ ksi

$D = 54$ in

$t_w = 0.50$ in

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

For the compression flange:

$P_c := F_{yc} \cdot b_c \cdot t_c$

Where:

F_{yc} = Specified minimum yield strength of a compression flange (ksi)

b_c = Full width of the compression flange (in)

t_c = Thickness of compression flange (in)

$F_{yc} := 50$ ksi

$b_c := 14$ in

$t_c := 0.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 525}$ kips

For the slab:

$P_s := 0.85 \cdot f'_c \cdot b_s \cdot t_s$

Where:

b_s = Effective width of concrete deck (in)

t_s = Thickness of concrete deck (in)

$f'_c = 4.00$ ksi

$b_s := 120$ in

$t_s = 8.5$ in

$P_s := 0.85 \cdot f'_c \cdot b_s \cdot t_s$ $\boxed{P_s = 3468}$ kips

The forces in the longitudinal reinforcement may be conservatively neglected in regions of positive flexure.

Check the location of the plastic neutral axis, as follows:

$\boxed{P_t + P_w = 1963}$	kips	$\boxed{P_c + P_s = 3993}$	kips
$\boxed{P_t + P_w + P_c = 2488}$	kips	$\boxed{P_s = 3468}$	kips

Since $P_t + P_w + P_c < P_s$, the plastic neutral axis is located within the slab **LRFD [Table D6.1-1]**. Since the slab reinforcement is being neglected in regions of positive flexure, Case III, V, or VII can be used. All three cases yield the same results with the reinforcement terms P_{rt} and P_{rb} set equal to zero.

$$Y := (t_s)\cdot\left(\frac{P_c + P_w + P_t}{P_s}\right) \qquad \boxed{Y = 6.10} \quad \text{in}$$

Check that the position of the plastic neutral axis, as computed above, results in an equilibrium condition in which there is no net axial force.

$$\text{Compression} := 0.85\cdot f'_c\cdot b_s\cdot Y \qquad \boxed{\text{Compression} = 2488} \quad \text{kips}$$

$$\text{Tension} := P_t + P_w + P_c \qquad \boxed{\text{Tension} = 2488} \quad \text{kips} \quad \text{OK}$$

The plastic moment, M_p, is computed as follows, where d is the distance from an element force (or element neutral axis) to the plastic neutral axis **LRFD [Table D6.1-1]**:

$$d_c := \frac{-t_c}{2} + 3.75 + t_s - Y \qquad \boxed{d_c = 5.78} \quad \text{in}$$

$$d_w := \frac{D}{2} + 3.75 + t_s - Y \qquad \boxed{d_w = 33.15} \quad \text{in}$$

$$d_t := \frac{t_t}{2} + D + 3.75 + t_s - Y \qquad \boxed{d_t = 60.59} \quad \text{in}$$

$$M_p := \frac{\dfrac{Y^2\cdot P_s}{2\cdot t_s} + \left(P_c\cdot d_c + P_w\cdot d_w + P_t\cdot d_t\right)}{12} \qquad \boxed{M_p = 7707} \quad \text{kip-ft}$$

E24-1.9 Determine if Section is Compact or Noncompact - Positive Moment Region

Since the section is in a straight bridge, the next step in the design process is to determine if the section is compact or noncompact. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

If the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the compact-section web slenderness provisions, as follows **LRFD [6.10.6.2.2]**:

$$\frac{2 \cdot D_{cp}}{t_w} \leq 3.76 \cdot \sqrt{\frac{E}{F_{yc}}}$$

Where:

D_{cp} = Depth of web in compression at the plastic moment (in)

Since the plastic neutral axis is located within the slab,

$D_{cp} := 0$ in

Therefore the web is deemed compact. Since this is a composite section in positive flexure and there are no holes in the tension flange at this section, the flexural resistance is computed as defined by the composite compact-section positive flexural resistance provisions of **LRFD [6.10.7.1.2]**.

E24-1.10 Design for Flexure - Strength Limit State - Positive Moment Region

Since the section was determined to be compact, and since it is a composite section in the positive moment region with no holes in the tension flange, the flexural resistance is computed in accordance with **LRFD [6.10.7.1.2]**.

$$M_n := 1.3 \cdot R_h \cdot M_y$$

Where:

R_h = Hybrid factor

M_y = Yield Moment (kip-in)

All design sections of this girder are homogenous. That is, the same structural steel is used for the top flange, the web, and the bottom flange. Therefore, the hybrid factor, R_h, is as follows **LRFD [6.10.1.10.1]**:

$R_h := 1.0$

The yield moment, M_y, is computed as follows **LRFD [Appendix D6.2.2]**:

$$F_y := \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

Where:

M_{D1} = Bending moment caused by the factored permanent load applied before the concrete deck has hardened or is made composite (kip-in)

S_{NC} = Noncomposite elastic section modulus (in^3)

M_{D2} = Bending moment caused by the factored permanent load applied to the long-term composite section (kip-in)

S_{LT} = Long-term composite elastic section modulus (in^3)

M_{AD} = Additional bending moment that must be applied to the short-term composite section to cause nominal yielding in either steel flange (kip-in)

S_{ST} = Short-term composite elastic section modulus (in^3)

$$M_y := M_{D1} + M_{D2} + M_{AD}$$

$F_y := 50$ ksi

$M_{D1} := (1.25 \cdot 1113.7)$ $\boxed{M_{D1} = 1392}$ kip-ft

$M_{D2} := (1.25 \cdot 159.1) + (1.50 \cdot 150.6)$ $\boxed{M_{D2} = 425}$ kip-ft

For the bottom flange:

$S_{NC} := 877.6$ in^3

$S_{LT} := 1231.4$ in^3

$S_{ST} := 1339.5$ in^3

$$M_{AD} := \left[\frac{S_{ST}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT}}{12^3}}\right)\right]$$ $\boxed{M_{AD} = 2994}$ kip-ft

$M_{ybot} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ybot} = 4811}$ kip-ft

For the top flange:

$S_{NC} := 821.7$ in^3

$S_{LT} := 4404.7$ in^3

$S_{ST} := 24820.6$ in^3

$$M_{AD} := \frac{S_{ST}}{12^3} \cdot \left(F_y \cdot 144 - \frac{M_{D1}}{\frac{S_{NC}}{12^3}} - \frac{M_{D2}}{\frac{S_{LT}}{12^3}}\right)$$ $\boxed{M_{AD} = 58974}$ kip-ft

$M_{ytop} := M_{D1} + M_{D2} + M_{AD}$ $\boxed{M_{ytop} = 60791}$ kip-ft

The yield moment, M_y, is the lesser value computed for both flanges. Therefore, M_y is determined as follows **LRFD [Appendix D6.2.2]**:

$M_y := \min(M_{ybot}, M_{ytop})$ $\boxed{M_y = 4811}$ kip-ft

Therefore, for the positive moment region of this design example, the nominal flexural resistance is computed as follows **LRFD [6.10.7.1.2]**:

$D_p \le 0.1D_t$

$D_p := Y$ $\boxed{D_p = 6.10}$ in

$D_t := 0.875 + 54 + .75 + 3 + 8.5$ $\boxed{D_t = 67.13}$ in

$\boxed{0.1 \cdot D_t = 6.713}$ in OK

Therefore

$M_n := M_p$ $\boxed{M_n = 7707}$ kip-ft

Since this is neither a simple span nor a continuous span where the span and the sections in the negative-flexure region over the interior supports satisfy the special conditions outlined at the end of **LRFD [6.10.7.1.2]**, the nominal flexural resistance of the section must not exceed the following:

$M_n := 1.3 \cdot R_h \cdot M_y$ $\boxed{M_n = 6255}$ kip-ft

The ductility requirement is checked as follows **LRFD [6.10.7.3]**:

$D_p \le 0.42D_t$

Where:

D_p = Distance from top of the concrete deck to the neutral axis of the composite section at the plastic moment (in)

D_t = Total depth of the composite section (in)

$\boxed{0.42 \cdot D_t = 28.19}$ in OK

The factored flexural resistance, M_r, is computed as follows (note that since there is no curvature, skew and wind load is not considered under the Strength I load combination, the flange lateral bending stress is taken as zero in this case **LRFD [6.10.7.1.1]**:

$$M_u + \frac{1}{3}(0) \le \phi_f M_n$$

Where:

M_u = Moment due to the factored loads (kip-in)

M_n = Nominal flexural resistance of a section (kip-in)

$\phi_f := 1.00$

$M_r := \phi_f \cdot M_n$ $\boxed{M_r = 6255}$ kip-ft

The positive flexural resistance at this design section is checked as follows:

$\Sigma\eta_i \cdot \gamma_i \cdot Q_i \leq R_r$

or in this case:

$\Sigma\eta \cdot \gamma \cdot M_u \leq M_r$

$\eta := 1.00$

As computed in E24-1.6,

$\Sigma\gamma \cdot M_u := 5183$ kip-ft

Therefore

$\Sigma\eta \cdot \gamma \cdot M_u := 5183$ kip-ft

$\boxed{M_r = 6255}$ kip-ft OK

E24-1.11 Design for Shear - Positive Moment Region

Shear must be checked at each section of the girder **LRFD [6.10.9]**. However, shear is minimal at the location of maximum positive moment, and it is maximum at the pier.

Therefore, for this design example, the required shear design computations will be presented later for the girder design section at the pier.

It should be noted that in end panels, the shear is limited to either the shear yield or shear buckling in order to provide an anchor for the tension field in adjacent interior panels. Tension field is not allowed in end panels. The design procedure for shear in the end panel is presented in **LRFD [6.10.9.3.3c]**.

E24-1.12 Design Transverse Intermediate Stiffeners - Positive Moment Region

As stated above, shear is minimal at the location of maximum positive moment but is maximum at the pier. Therefore, the required design computations for transverse intermediate stiffeners will be presented later for the girder design section at the pier **LRFD [6.10.11.1]**.

E24-1.13 Design for Flexure - Fatigue and Fracture Limit State - Positive Moment Region

Load-induced fatigue must be considered in a plate girder design **LRFD [6.6.1]**.

For this design example, fatigue will be checked for the fillet-welded connection of a transverse intermediate stiffener serving as a cross-frame connection plate to the girder at the location of maximum positive moment. This detail corresponds to Description 4.1 in **LRFD [Table 6.6.1.2.3-1]**, and it is classified as Detail Category C'. The fatigue detail at the inner fiber of the tension flange, where the transverse intermediate stiffener is welded to the flange, is subject to a net tensile stress by inspection. However, for simplicity, the computations will conservatively compute the fatigue stress at the outer fiber of the tension flange.

The fatigue detail being investigated in this design example is illustrated in the following figure:

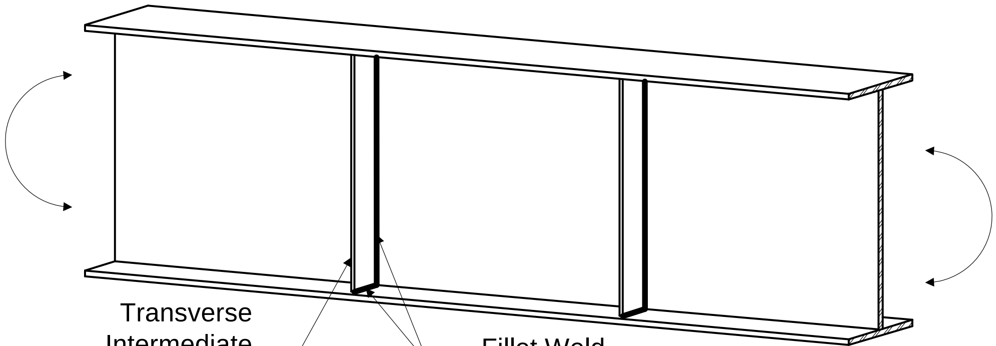


Figure E24-1.13-1
Load-Induced Fatigue Detail

The nominal fatigue resistance is computed as follows **LRFD [6.6.1.2.5]**:

NOTE: WisDOT policy is to design for infinite fatigue life (ADTT not considered) and use Fatigue I limit state.

$\Delta F_n := \Delta F_{TH}$

Where:

ΔF_{TH} = Constant-amplitude fatigue threshold **LRFD [Table 6.6.1.2.5-3]** (ksi)

$\Delta F_{TH} := 12.00$ ksi

$\Delta F_n = \blacksquare \cdot 12.00$ ksi

The factored fatigue stress range in the outer fiber base metal at the weld at the location of maximum positive moment was previously computed in Table E24-1.6-1, as follows:

$f_{botgdr} := 9.90$ ksi

$f_{botgdr} \le \Delta F_n$ OK

In addition to the above fatigue detail check, a special fatigue requirement for webs must also be checked **LRFD [6.10.6]**. These calculations will be presented later for the girder design section at the pier [E24-1.23].

E24-1.14 Design for Flexure - Service Limit State - Positive Moment Region

The girder must be checked for service limit state control of permanent deflection **LRFD [6.10.4.2]**. This check is intended to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. The Service II load combination is used for this check.

The stresses for steel flanges of composite sections must satisfy the following requirements **LRFD [6.10.4.2.2]**:

Top flange:

$$f_f \leq 0.95R_h \cdot F_{yf}$$

Bottom flange

$$f_f + \frac{f_l}{2} \leq 0.95R_h \cdot F_{yf}$$

Since there is no curvature and no discontinuous diaphragm lines in conjunction with skews exceeding 20 degrees, f_l is taken equal to zero at the service limit state in this case. The factored Service II flexural stress was previously computed in Table E24-1.6-1 as follows:

$f_{botgdr} := 40.65$ ksi

$f_{topgdr} := -18.32$ ksi

$\boxed{0.95 \cdot R_h \cdot F_{yf} = 47.50}$ ksi OK

As indicated in **LRFD [6.10.4.2.2]**, the web bend buckling check at the service limit state must be checked for all sections according to equation 6.10.4.2.2-4 with the execption of composite sections in positive flexure that meet the requirement of **LRFD [6.10.2.1.1]** (D/tw ≤ 150). Since $\frac{D}{t_w} = 108$ [E24-1.7], equation 6.10.4.2.2-4 does not need to be considered for this location.

In addition to the check for service limit state control of permanent deflection, the girder can also be checked for live load deflection **LRFD [2.5.2.6.2]**. Although this check is optional for a concrete deck on steel girders, it is included in this design example.

Using an analysis computer program, the maximum live load deflection is computed to be the following:

$\Delta_{max} := 1.14$ in

This maximum live load deflection is computed based on the following:

1. All design lanes are loaded.
2. All supporting components are assumed to deflect equally.
3. For composite design, the design cross section includes the entire width of the roadway.
4. The number and position of loaded lanes is selected to provide the worst effect.
5. The live load portion of Service I Limit State is used.
6. Dynamic load allowance is included.
7. The live load is taken from **LRFD [3.6.1.3.2]**.

As recommended in LRFD [2.5.2.6.2] for "vehicular load, general", the deflection limit is as follows:

$Span := 120$ ft

$\Delta_{allowable} := \left(\frac{Span}{800}\right)\cdot(12)$ $\boxed{\Delta_{allowable} = 1.80}$ in OK

E24-1.15 Design for Flexure - Constructibility Check - Positive Moment Region

The girder must also be checked for flexure during construction **LRFD [6.10.3.2]**. The girder has already been checked in its final condition when it behaves as a composite section. The constructibility must also be checked for the girder prior to the hardening of the concrete deck when the girder behaves as a noncomposite section.

As previously stated, a deck pouring sequence will not be considered in this design example. However, it is required to consider the effects of the deck pouring sequence in an actual design because it will often control the design of the top flange and the cross-frame spacing in the positive moment regions of composite girders. The calculations illustrated below, which are based on the final noncomposite dead load moments after the sequential placement is complete would be employed to check the girder for the critical actions resulting from the deck pouring sequence. For an exterior girder, deck overhang effects must also be considered according to **LRFD [6.10.3.4]**. Since an interior girder is designed in this example, those effects are not considered here.

Based on the flowchart for constructibility checks in **LRFD [Appendix C6]**, nominal yielding of both flanges must be checked as well as the flexural resistance of the compression flange. For discretely braced flanges (note f_l is taken as zero since this is an interior girder and there are no curvature, skew, deck overhang or wind load effects considered) **LRFD [6.10.3.2.1 & 6.10.3.2.2]**:

$$f_{bu} + f_l \le \phi_f \cdot R_h \cdot F_{yf}$$

The flange stress, f_{bu}, is taken from Table E24-1.6-1 for the noncomposite dead load for the top flange since no deck placement analysis was performed. By inspection, since lateral flange bending is not considered, and no live load effects are considered, Strength IV is the controlling limit state and the compression flange is the controlling flange.

$f_{bu} := 1.5\cdot16.26$ ksi $\boxed{f_{bu} = 24.39}$ ksi

$\boxed{\phi_f\cdot R_h\cdot F_{yf} = 50.00}$ ksi OK

The flexural resistance calculation ensures that the compression flange has sufficient strength with respect to lateral torsional and flange local buckling based limit states, including the consideration of flange lateral bending where these effects are judged to be significant. The equation is in **LRFD [6.10.3.2]**:

$$f_{bu} + \frac{1}{3}\cdot f_l \le \phi_f \cdot F_{nc}$$

Where:

F_{nc} = Nominal flexural resistance of the flange (ksi)

For straight I-girder bridges with compact or noncompact webs, the nominal resistance may be calculated from **LRFD [Appendix A6.3.3]** which includes the beneficial contribution of the St. Venant constant, J, in the calculation of the lateral torsional buckling resistance. This example will not use **LRFD [Appendix A6.3.3]**, but a check of the noncompact slenderness limit of web using **LRFD [6.10.6.2.3]** is included for reference.

$D_c := 28.73 - 0.75$ $\boxed{D_c = 27.98}$ in

$\boxed{\frac{2 \cdot D_c}{t_w} = 111.92}$

$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E_s}{F_{yc}}}$ $\boxed{\lambda_{rw} = 137.27}$

$\frac{2 \cdot D_c}{t_w} < 5.7 \cdot \sqrt{\frac{E}{F_{yc}}}$ OK

Although the noncomposite section has a nonslender web according to equation 1 of **LRFD [6.10.6.2.3]**, for this example, these beneficial effects will conservatively not be utilized.

The nominal flexural resistance of the compression flange is therefore taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance calculated according to **LRFD [6.10.8.2]**.

Local buckling resistance **LRFD [6.10.8.2.2]**:

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}}$$

Where:

λ_f	= Slenderness ratio for the compression flange
b_{fc}	= Full width of the compression flange (in)
t_{fc}	= Thickness of the compression flange (in)

$b_{fc} := 14$ in (see Figure E24-1.2-1)

$t_{fc} := 0.75$ in (see Figure E24-1.2-1)

$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}}$ $\boxed{\lambda_f = 9.33}$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_{yc}}}$$

Where:

λ_{pf}	= Limiting slenderness ratio for a compact flange

$\boxed{\lambda_{pf} = 9.15}$

Since $\lambda_f > \lambda_{pf}$, F_{nc} must be calculated by the following equation:

$$F_{nc} := \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

Where:

F_{yr}	= Compression-flange stress at the onset of nominal yielding within the cross-section, including residual stress effects, but not including compression-flange lateral bending, taken as the smaller of $0.7F_{yc}$ and F_{yw}, but not less than $0.5F_{yc}$
λ_{rf}	= Limiting slenderness ratio for a noncompact flange
R_b	= Web load-shedding factor **LRFD [6.10.1.10.2]**

$F_{yr} := \max(\min(0.7 \cdot F_{yc}, F_{yw}), 0.5 \cdot F_{yc})$ $\boxed{F_{yr} = 35.00}$ ksi

$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_s}{F_{yr}}}$ $\boxed{\lambda_{rf} = 16.12}$

$R_b := 1.0$

$$F_{nc} := \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$\boxed{F_{nc} = 49.61}$ ksi

Lateral torsional buckling resistance **LRFD [6.10.8.2.3]**:

For the noncomposite loads during construction:

$Depth_{comp} := 55.625 - 26.897$ (see Figure E24-1.2-1 and Table E24-1.3-1)

$\boxed{Depth_{comp} = 28.73}$ in

The effective radius of gyration, r_t, for lateral torsional buckling is calculated as follows:

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}}$$

Where:

D_c	= Depth of the web in compression in the elastic range (in). For composite sections see **LRFD [Appendix D6.3.1]**

$t_{topfl} := 0.75$ in

$D_c := Depth_{comp} - t_{topfl}$ $\boxed{D_c = 27.98}$ in

$$r_t := \frac{b_{fc}}{\sqrt{12\cdot\left(1+\frac{1}{3}\cdot\frac{D_c\cdot t_w}{b_{fc}\cdot t_{fc}}\right)}}$$ $\boxed{r_t = 3.36}$ in

The limiting unbraced length, L_p, to achieve the nominal flexural resistance of $R_bR_hF_{yc}$ under uniform bending is calculated as follows:

$$L_p := 1.0\cdot r_t\cdot\sqrt{\frac{E_s}{F_{yc}}}$$ $\boxed{L_p = 80.99}$ in

The limiting unbraced length, L_r, to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression-flange residual stress effects is calculated as follows:

$$L_r := \pi\cdot r_t\cdot\sqrt{\frac{E_s}{F_{yr}}}$$ $\boxed{L_r = 304.13}$ in

$L_b = 240.00$ in

The moment gradient correction factor, C_b, is computed as follows:

Note since f_{mid} is greater than f_2 at the location of maximum positive moment (see Figure E24-1.1-3), use C_b = 1.0 according to **LRFD [6.10.8.2.3]**.

$C_b := 1.00$

Therefore:

$$F_{nc} := C_b\cdot\left[1-\left(1-\frac{F_{yr}}{R_h\cdot F_{yc}}\right)\cdot\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]\cdot R_b\cdot R_h\cdot F_{yc}$$

$\boxed{F_{nc} = 39.3}$ ksi

Use

(minimum of local buckling and lateral torsional buckling) $F_{nc} := 39.3$ ksi

$\boxed{\phi_f\cdot F_{nc} = 39.30}$ ksi

$\boxed{f_{bu}+\frac{1}{3}\cdot(0) = 24.39}$ ksi OK

Web bend-buckling during construction must also be checked according to equation 3 of **LRFD [6.10.3.2.1]**. However, since the noncomposite section has previously been shown to have a nonslender web, web bend-buckling need not be checked in this case according to **LRFD [6.10.3.2.1]**.

In addition to checking the nominal flexural resistance during construction, the nominal shear resistance must also be checked **LRFD [6.10.3.2.3]**. However, shear is minimal at the location of maximum positive moment, and it is maximum at the pier in this case.

Therefore, for this design example, the nominal shear resistance for constructibility will be presented later for the girder design section at the pier.

E24-1.16 - Check Wind Effects on Girder Flanges - Positive Moment Region

As stated in previously, for this design example, the interior girder is being designed.

Wind effects generally do not control a steel girder design, and they are generally considered for the exterior girders only **LRFD [6.10.1.6 & C4.6.2.7.1]**. However, for this design example, wind effects will be presented later for the girder design section at the pier for illustration only.

Specification checks have been completed for the location of maximum positive moment, which is at 0.4L in Span 1.

E24-1.17 Check Section Proportion Limits - Negative Moment Region

Now the specification checks are repeated for the location of maximum negative moment, which is at the pier, as shown in Figure E24-1.17-1. This is also the location of maximum shear in this case.

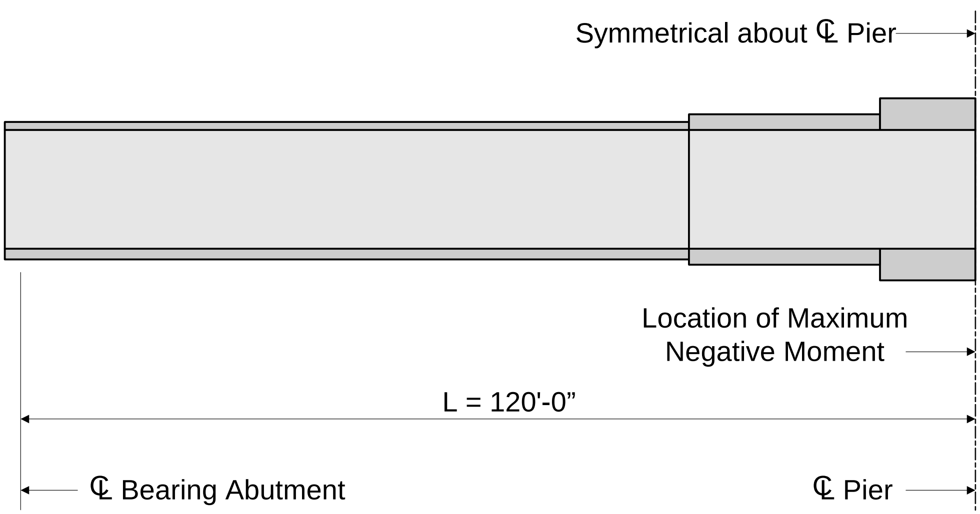


Figure E24-1.17-1
Location of Maximum Negative Moment

Several checks are required to ensure that the proportions of the girder section are within specified limits **LRFD [6.10.2]**.

The first section proportion check relates to the web slenderness **LRFD [6.10.2.1]**. For a section without longitudinal stiffeners, the web must be proportioned such that:

$$\frac{D}{t_w} \le 150 \qquad \boxed{\frac{D}{t_w} = 108.00} \qquad \text{OK}$$

The second set of section proportion checks relate to the general proportions of the section

LRFD [6.10.2.2]. The compression and tension flanges must be proportioned such that:

$$\frac{b_f}{2 \cdot t_f} \le 12.0$$

$b_f := 14$

$t_f := 2.50$

$\boxed{\frac{b_f}{2 \cdot t_f} = 2.80}$ OK

$b_f \ge \frac{D}{6}$ $\boxed{\frac{D}{6} = 9.00}$ in OK

$t_f \ge 1.1 \cdot t_w$ $\boxed{1.1t_w = 0.55}$ in OK

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10$$

$I_{yc} := \frac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in^4

$I_{yt} := \frac{2.50 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in^4

$\boxed{\frac{I_{yc}}{I_{yt}} = 1.100}$ OK

E24-1.18 Compute Plastic Moment Capacity - Negative Moment Region

For composite sections, the plastic moment, M_p, is calculated as the first moment of plastic forces about the plastic neutral axis **LRFD [Appendix D6.1]**. For composite sections in negative flexure, the concrete deck is ignored and the longitudinal deck reinforcement is included in the computation of M_p.

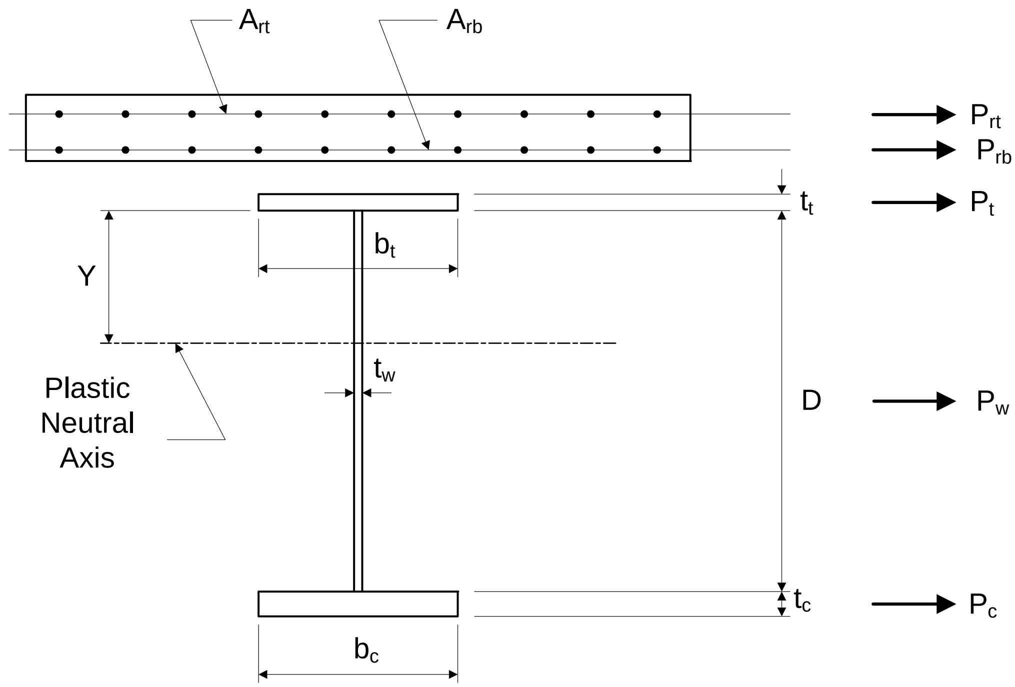


Figure E24-1.18-1
Computation of Plastic Moment Capacity for Negative Bending Sections

The plastic force in the tension flange, P_t, is calculated as follows:

$t_t := 2.50$ in

$P_t := F_{yt} \cdot b_t \cdot t_t$ $\boxed{P_t = 1750}$ kips

The plastic force in the web, P_w, is calculated as follows:

$P_w := F_{yw} \cdot D \cdot t_w$ $\boxed{P_w = 1350}$ kips

The plastic force in the compression flange, P_c, is calculated as follows:

$t_c := 2.75$ in

$P_c := F_{yc} \cdot b_c \cdot t_c$ $\boxed{P_c = 1925}$ kips

The plastic force in the top layer of longitudinal deck reinforcement, P_{rt}, used to compute the plastic moment is calculated as follows:

$P_{rt} := F_{yrt} \cdot A_{rt}$

Where:

F_{yrt} = Specified minimum yield strength of the top layer of longitudinal concrete deck reinforcement (ksi)

A_{rt} = Area of the top layer of longitudinal reinforcement within the effective concrete deck width (in^2)

$F_{yrt} := 60$ ksi

$A_{rt} := 0.44 \cdot \left(\frac{W_{effflange} \cdot 12}{7.5} \right)$ $\boxed{A_{rt} = 7.04}$ in^2

$P_{rt} := F_{yrt} \cdot A_{rt}$ $\boxed{P_{rt} = 422}$ kips

This example conservatively ignores the contribution from the bottom layer of longitudinal deck reinforcement, but the calculation is included for reference. The plastic force in the bottom layer of longitudinal deck reinforcement, P_{rb}, used to compute the plastic moment is calculated as follows:

$P_{rb} := F_{yrb} \cdot A_{rb}$

Where:

F_{yrb} = Specified minimum yield strength of the bottom layer of longitudinal concrete deck reinforcement (ksi)

A_{rb} = Area of the bottom layer of longitudinal reinforcement within the effective concrete deck width (in^2)

$F_{yrb} := 60$ ksi

$A_{rb} := 0 \cdot \left(\frac{W_{effflange} \cdot 12}{1} \right)$ $\boxed{A_{rb} = 0.00}$ in^2

$P_{rb} := F_{yrb} \cdot A_{rb}$ $\boxed{P_{rb} = 0}$ kips

Check the location of the plastic neutral axis, as follows:

$\boxed{P_c + P_w = 3275}$ kips

$\boxed{P_t + P_{rb} + P_{rt} = 2172}$ kips

$\boxed{P_c + P_w + P_t = 5025}$ kips

$\boxed{P_{rb} + P_{rt} = 422}$ kips

Therefore the plastic neutral axis is located within the web **LRFD [Appendix Table D6.1-2]**.

$Y := \left(\frac{D}{2} \right) \cdot \left(\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1 \right)$ $\boxed{Y = 22.05}$ in

Although it will be shown in the next design step that this section qualifies as a nonslender web section at the strength limit state, the optional provisions of Appendix A to **LRFD [6]** are not employed in this example. Thus, the plastic moment is not used to compute the flexural resistance and therefore does not need to be computed.

E24-1.19 Determine if Section is a Compact-Web, Noncompact-Web, or Slender-Web Section - Negative Moment Region

Since the section is in a straight bridge, the next step in the design process is to determine if the section is a compact-web, noncompact-web, or slender-web section. This, in turn, will determine which formulae should be used to compute the flexural capacity of the girder.

Where the specified minimum yield strengths of the flanges do not exceed 70.0 ksi and the girder does not have longitudinal stiffeners, then the first step is to check the noncompact-web slenderness limit, as follows **LRFD [6.10.6.2.3]**:

$$\frac{2 \cdot D_c}{t_w} \le 5.7\sqrt{\frac{E_s}{F_{yc}}} \qquad \lambda_{rw} := 5.7\sqrt{\frac{E_s}{F_{yc}}}$$

At sections in negative flexure, D_c of the composite section consisting of the steel section plus the longitudinal reinforcement is to be used at the strength limit state.

$D_c := 31.077 - 2.75$ (see Figure E24-1.2-1 and Table E24-1.3-3)

$\boxed{D_c = 28.33}$ in

$\boxed{\frac{2 \cdot D_c}{t_w} = 113.3}$

$\boxed{5.7 \cdot \sqrt{\frac{E_s}{F_{yc}}} = 137.3}$

The section is a nonslender web section (i.e. either a compact-web or noncompact-web section). Next, check:

$I_{yc} := \frac{2.75 \cdot 14^3}{12}$ $\boxed{I_{yc} = 628.83}$ in^4

$I_{yt} := \frac{2.5 \cdot 14^3}{12}$ $\boxed{I_{yt} = 571.67}$ in^4

$\boxed{\frac{I_{yc}}{I_{yt}} = 1.10}$ > 0.3 OK

Therefore, the web qualifies to use the optional provisions of **LRFD [Appendix A6]** to compute the flexural resistance. However, since the web slenderness is closer to the noncompact web slenderness limit than the compact web slenderness limit in this case, the simpler equations of **LRFD [6.10.8]**, which assume slender-web behavior and limit the resistance to Fyc or below, will conservatively be applied in this example to compute the flexural resistance at the strength limit state. The investigation proceeds by calculating the flexural resistance of the discretely braced compression flange.

E24-1.20 Design for Flexure - Strength Limit State - Negative Moment Region

The nominal flexural resistance of the compression flange shall be taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance **LRFD [6.10.8.2.2 & 6.10.8.2.3]**.

Local buckling resistance **LRFD [6.10.8.2.2]**:

$b_{fc} := 14$ (see Figure E24-1.2-1)

$t_{fc} := 2.75$ (see Figure E24-1.2-1)

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}} \qquad \boxed{\lambda_f = 2.55}$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_{yc}}} \qquad \boxed{\lambda_{pf} = 9.15}$$

Since $\lambda_f < \lambda_{pf}$, F_{nc} is calculated using the following equation:

$$F_{nc} := R_b \cdot R_h \cdot F_{yc}$$

Since $2D_c/t_w$ is less than λ_{rw} (calculated above), R_b is taken as 1.0 **LRFD [6.10.1.10.2]**.

$$\boxed{F_{nc} = 50.00} \quad \text{ksi}$$

Lateral torsional buckling resistance **LRFD [6.10.8.2.3]**:

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}} \qquad \boxed{r_t = 3.81} \quad \text{in}$$

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E_s}{F_{yc}}} \qquad \boxed{L_p = 91.86} \quad \text{in}$$

$$L_r := \pi \cdot r_t \cdot \sqrt{\frac{E_s}{F_{yr}}} \qquad \boxed{L_r = 344.93} \quad \text{in}$$

$$L_b = 240.00$$

The moment gradient correction factor, C_b, is computed as follows:

Where the variation in the moment along the entire length between brace points is concave in shape, which is the case here, $f_1 = f_0$. (calculated below based on the definition of f_0 given in **LRFD [6.10.8.2.3]**).

$M_{NCDC0.8L} := 112.1 + 780.3 + 17.8$ $\boxed{M_{NCDC0.8L} = 910.20}$ kip-ft

$S_{NCDC0.8L} := 2278.2$ in^3

$M_{par0.8L} := 110.3$ kip-ft

The section properties specified for the 0.8 pt are the properties found at the pier based on **LRFD [6.10.8.2.3]**.

$M_{fws0.8L} := 104.4$ kip-ft

$M_{LL0.8L} := 1165.9$ kip-ft

$S_{rebar0.8L} := 2380.2$ in^3

$$f_1 := 1.25 \cdot \frac{M_{NCDC0.8L} \cdot 12}{S_{NCDC0.8L}} + 1.25 \cdot \frac{M_{par0.8L} \cdot 12}{S_{rebar0.8L}} + 1.50 \cdot \frac{M_{fws0.8L} \cdot 12}{S_{rebar0.8L}} + 1.75 \cdot \frac{M_{LL0.8L} \cdot 12}{S_{rebar0.8L}}$$

$\boxed{f_1 = 17.76}$ ksi

$f_2 := 48.84$ ksi (Table E24-1.6-2)

$\boxed{\frac{f_1}{f_2} = 0.36}$

$C_b := 1.75 - 1.05 \cdot \left(\frac{f_1}{f_2}\right) + 0.3 \cdot \left(\frac{f_1}{f_2}\right)^2$ < 2.3 $\boxed{C_b = 1.41}$

Therefore:

$F_{yr} := \max\left(\min\left(0.7 \cdot F_{yc}, F_{yw}\right), 0.5 \cdot F_{yc}\right)$ $\boxed{F_{yr} = 35.00}$ ksi

$$F_{nc} := C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$\boxed{F_{nc} = 58.03}$ ksi

$F_{nc} \le R_b \cdot R_h \cdot F_{yc}$ $\boxed{R_b \cdot R_h \cdot F_{yc} = 50.00}$ ksi

Use:

$F_{nc} := 50.00$ ksi

$\boxed{\phi_f \cdot F_{nc} = 50.00}$ ksi

$f_{bu} := 48.84$ ksi (Table E24-1.6-2)

Since there are no curvature or skew effects and wind is not considered under the Strength I load combination, f_l is taken equal to zero. Therefore:

$\boxed{f_{bu} + \frac{1}{3} \cdot (0) = 48.84}$ ksi OK

The investigation proceeds by calculating the flexural resistance of the continuously braced tension flange **LRFD [6.10.8.1.3 & 6.10.8.3]**.

$f_{bu} \leq \phi_f \cdot R_h \cdot F_{yf}$ $\boxed{\phi_f \cdot R_h \cdot F_{yf} = 50.00}$ ksi

(Table E24-1.6-2) $f_{bu} := 47.52$ ksi OK

E24-1.21 - Design for Shear - Negative Moment Region

Shear must be checked at each section of the girder. For this design example, shear is maximum at the pier.

The first step in the design for shear is to check if the web must be stiffened. The nominal shear resistance, V_n, of unstiffened webs of hybrid and homogeneous girders is **LRFD [6.10.9.2]**:

$$V_n := C \cdot V_p$$

Where:

C = Ratio of the shear-buckling resistance to the shear yield strength in accordance with **LRFD [6.10.9.3.2]**, with the shear-buckling coefficient, k, taken equal to 5.0

V_p = Plastic shear force (kips)

$k := 5.0$ $\boxed{\frac{D}{t_w} = 108.00}$

$$\boxed{1.12 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 60.31}$$

$$\boxed{1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 75.39}$$

Therefore,

$$\frac{D}{t_w} \geq 1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}}$$

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E_s \cdot k}{F_{yw}}\right) \qquad \boxed{C = 0.390}$$

The plastic shear force, V_p, is then:

$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w$ $\boxed{V_p = 783.0}$ kips

$V_n := C \cdot V_p$ $\boxed{V_n = 305.6}$ kips

The factored shear resistance, V_r, is computed as follows **LRFD [6.10.9.1]**:

$\phi_v := 1.00$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 305.6}$ kips

The shear resistance at this design section is checked as follows:

$\Sigma\eta_i \cdot \gamma_i \cdot Q_i \leq R_r$

Or in this case:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i \leq V_r$

$\eta_i := 1.00$

As computed in E24-1.6, the factored Strength I Limit State shear at the pier is as follows:

$\Sigma\eta_i \cdot \gamma_i \cdot V_i := 414.3$ kips

$\boxed{V_r = 305.6}$ kips

Since the shear resistance of an unstiffened web is less than the actual design shear, the web must be stiffened.

The transverse intermediate stiffener spacing is 120 inches. The spacing of the transverse intermediate stiffeners does not exceed 3D, therefore the design section can be considered stiffened and the provisions of **LRFD [6.10.9.3]** apply.

The section must be checked against the web to flange proportion limits for interior web panels **LRFD [6.10.9.3.2]**.

$$\frac{2 \cdot D \cdot t_w}{b_{fc} \cdot t_{fc} + b_{ft} \cdot t_{ft}} \leq 2.5$$

Where:

b_{ft} = Full width of tension flange (in)

t_{ft} = Thickness of tension flange (in)

$b_{ft} := 14.0$

$t_{ft} := 2.50$

$$\boxed{\frac{2 \cdot D \cdot t_w}{b_{fc} \cdot t_{fc} + b_{ft} \cdot t_{ft}} = 0.73}$$ OK

The nominal shear resistance , V_n, of the interior web panel at the pier is then:

$$V_n := \left[V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}}\right]\right]$$

Where:

C = Ratio of the shear-buckling resistance to the shear yield strength

d_0 = Transverse stiffener spacing (in)

$d_o := 120$

$$k := 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2}$$ $\boxed{k = 6.01}$

$\boxed{\frac{D}{t_w} = 108.00}$

$\boxed{1.12 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 66.14}$

$\boxed{1.40 \cdot \sqrt{\frac{E_s \cdot k}{F_{yw}}} = 82.67}$

$$\frac{D}{t_w} \geq 1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}}$$

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E_s \cdot k}{F_{yw}}\right)$$ $\boxed{C = 0.469}$

$V_p = 783.00$

$$V_n := V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}}\right]$$ $\boxed{V_n = 515.86}$ kips

The factored shear resistance, V_r, is computed as follows:

$\phi_v := 1.00$

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 515.86}$ kips

As previously computed, for this design example:

$\Sigma \eta_i \cdot \gamma_i \cdot V_i := 414.3$ kips

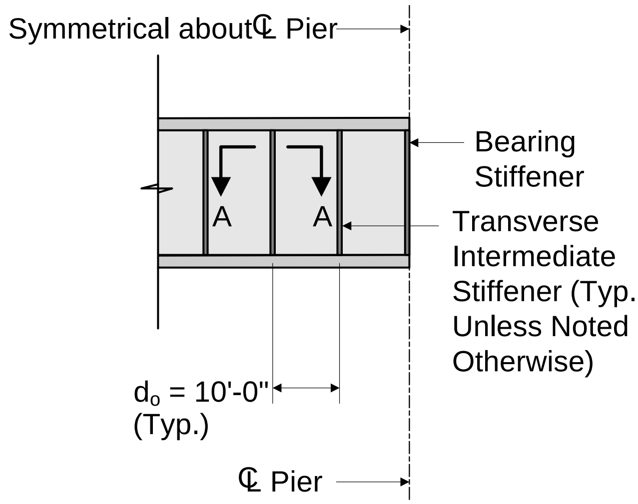
$\boxed{V_r = 515.86}$ kips OK

Therefore, the girder design section at the pier satisfies the shear resistance requirements for the web.

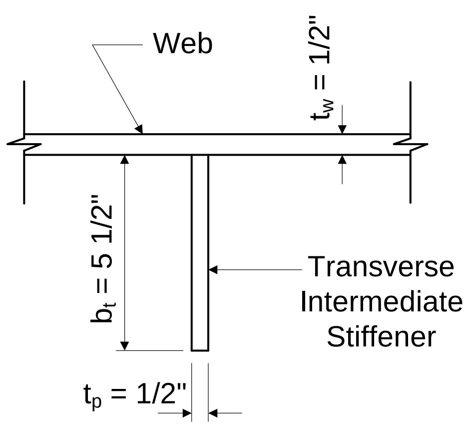
E24-1.22 Design Transverse Intermediate Stiffeners - Negative Moment Region

It is assumed that the transverse intermediate stiffeners consist of plates welded to one side of the web. The required interface between the transverse intermediate stiffeners and the top and bottom flanges is described in **LRFD [6.10.11.1.1]**.

The transverse intermediate stiffener configuration is assumed to be as presented in the following figure.



Partial Girder Elevation at Pier



Section A-A

Figure E24-1.22-1
Transverse Intermediate Stiffener

The first specification check is for the projecting width of the transverse intermediate stiffener. The width, b_t, of each projecting stiffener element must satisfy the following **LRFD [6.10.11.1.2]**:

$b_t \geq 2.0 + \dfrac{D}{30.0}$ and $16.0 \cdot t_p \geq b_t \geq 0.25 b_f$

Where:

t_p = Thickness of the projecting stiffener element (in)

b_f = Full width of the widest compression flange within the field section under consideration (in)

$b_t := 5.5$ in

$D := 54$ in

$t_p := 0.50$ in

$b_f = 14.00$ in

$\boxed{2.0 + \dfrac{D}{30.0} = 3.80}$ in OK

$\boxed{16.0 \cdot t_p = 8.00}$ in

$\boxed{0.25 \cdot b_f = 3.50}$ in OK

The moment of inertia, I_t, of the transverse stiffener must satisfy the following since each panel adjacent to the stiffener supports a shear force larger than the shear buckling resistance ($V_{cr} = CV_p$) **LRFD [6.10.11.1.3]**:

If $I_{t2} > I_{t1}$, then :

$$I_t \geq I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right)$$

Otherwise:

$$I_t \geq I_{t2}$$

$$I_{t1} := b \cdot t_w^{\,3} \cdot J$$

Where:

b = The smaller of do and D (in)

J = Stiffener bending rigidity parameter

$b := \min\left(d_o, D\right)$ $b = 54.00$ in

$$J := \max\left[\frac{2.5}{\left(\frac{d_o}{D}\right)^2} - 2.0, 0.5\right] \qquad J = 0.50$$

$$I_{t1} := b \cdot t_w^{\,3} \cdot J = 3.38 \qquad \text{in}^4$$

$$I_{t2} := \frac{D^4 \cdot \rho_t^{\,1.3}}{40} \cdot \left(\frac{F_{yw}}{E}\right)^{1.5}$$

Where:

ρ_t = The larger of F_{yw}/F_{crs} and 1.0

The local buckling stress for the stiffener, F_{crs}, is calculated as follows:

$$F_{crs} := \frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2} \le F_{ys}$$

Where:

F_{ys} = Specified minimum yield strength of the stiffener (ksi)

$$\boxed{\frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2} = 74.30} \qquad \text{ksi}$$

$$F_{ys} := 50.00 \qquad \text{ksi}$$

Use

$$F_{crs} := \min\left[F_{ys}, \frac{0.31 \cdot E_s}{\left(\frac{b_t}{t_p}\right)^2}\right] \qquad \boxed{F_{crs} = 50.00} \qquad \text{ksi}$$

$$\rho_t := \max\left(\frac{F_{yw}}{F_{crs}}, 1.0\right) \qquad \boxed{\rho_t = 1.00}$$

$$\boxed{I_{t2} := \frac{D^4 \cdot \rho_t^{\,1.3}}{40} \cdot \left(\frac{F_{yw}}{E_s}\right)^{1.5} = 15.22} \qquad \text{in}^4$$

Since $I_{t2} > I_{t1}$, the moment of inertia, I_t, of the transverse stiffener must satisfy:

$$I_t \geq I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right)$$

$V_u := 414.3$ kip

$V_{cr} := C \cdot V_p = 367.53$ kip

$V_n = 515.86$ kip

$I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right) = 7.11$ in^4

$I_t := \frac{t_p \cdot b_t^3}{3}$ $\boxed{I_t = 27.73}$ in^4

Therefore,

$$I_t \geq I_{t1} + \left(I_{t2} - I_{t1}\right)\left(\frac{V_u - \phi_v \cdot V_{cr}}{\phi_v \cdot V_n - \phi_v \cdot V_{cr}}\right) \quad \text{OK}$$

E24-1.23 Design for Flexure - Fatigue and Fracture Limit State - Negative Moment Region

For this design example, sample nominal fatigue resistance computations were presented previously (E24-1.13) for the girder section at the location of maximum positive moment **LRFD [6.6.1]**. Detail categories are explained and illustrated in **LRFD [Table 6.6.1.2.3-1]**.

In addition to the nominal fatigue resistance computations, a special fatigue requirement for webs must also be checked **LRFD [6.10.5.3]**. This check is required to control out-of-plane flexing of the web due to shear under repeated live loading.

The check is made using fatigue range live load shear in combination with the shear due to the unfactored permanent load. This total shear is limited to the shear buckling resistance ($V_{cr} = CV_p$), as follows:

$$V_u \leq V_{cr}$$

Based on the unfactored shear values in Table E24-1.6-3:

$V_u = V_{noncomp} + V_{par} + V_{fws} + 1.5V_{LLfatiguerange}$

$V_u := 111.5 + 14.5 + 13.8 + (1.5 \cdot 47.4)$ $\boxed{V_u = 210.90}$ kips

$C = 0.469$ See E24-1.21

$V_p = 783.00$ kips See E24-1.21

$V_{cr} := C \cdot V_p$ $\boxed{V_{cr} = 367.53}$ kips

$V_u \le V_{cr}$ OK

Therefore, the special fatigue requirement for webs for shear is satisfied.

Other fatigue resistance calculations in the negative moment region are not shown here, but would be similar to the sample check illustrated previously for the positive moment region (E24-1.13).

E24-1.24 Design for Flexure - Service Limit State - Negative Moment Region

The girder must be checked for service limit state control of permanent deflection **LRFD [6.10.4]**. Service II Limit State is used for this check.

The flange stress checks of **LRFD [6.10.4.2.2]** will not control for composite sections in negative flexure for which the nominal flexural resistance under the strength load combinations given in **LRFD [Table 3.4.1-1]** is determined according to the slender-web provision of **LRFD [6.10.8]**, which is the case in this example.

However, for sections in negative flexure, the web must satisfy the web bend buckling check given by equation 4 of **LRFD [6.10.4.2.2]** at the service limit state, using the appropriate value of the depth of the web in compression in the elastic range, D_c.

$$f_c \le F_{crw}$$

$$F_{crw} := \frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2} \qquad \text{(LRFD 6.10.1.9.1-1)}$$

Where:

k = Bend-buckling coefficient = $9/(D_c/D)^2$

The factored Service II flexural stress was previously computed in Table E24-1.6-2 as follows:

$f_{botgdr} := -34.22$ ksi

$f_{topgdr} := 22.39$ ksi

As previously explained, for this design example, the concrete slab is assumed to be fully effective for both positive and negative flexure for service limit states. Therefore, when this assumption is made, D_c must be computed as follows as indicated in **LRFD [Appendix D6.3.1]**:

$$D_c := \left(\frac{-f_c}{|f_c| + f_t}\right) \cdot d - t_{fc} \ge 0$$

$Depth_{gdr} := 59.25$ in (see Figure E24-1.2-1)

$$Depth_{comp} := \frac{-f_{botgdr}}{|f_{botgdr}| + f_{topgdr}} \cdot Depth_{gdr} \qquad \boxed{Depth_{comp} = 35.82} \text{ in}$$

$t_{botfl} := 2.75$ in

$D_c := Depth_{comp} - t_{botfl}$ $\boxed{D_c = 33.07}$ in

$D := 54.0$ in

$$k := \frac{9.0}{\left(\frac{D_c}{D}\right)^2}$$ $\boxed{k = 24.00}$

$$\boxed{\frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2} = 53.71}$$ ksi

$$F_{crw} := \min\left[\frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2}, R_h \cdot F_{yc}, \frac{F_{yw}}{0.7}\right]$$ $\boxed{F_{crw} = 50.00}$ ksi

$t_{bf} := 2.75$ in

$$f_c := f_{botgdr} \cdot \left(\frac{D_c}{D_c + t_{bf}}\right)$$ $\boxed{f_c = -31.59}$ ksi OK

E24-1.25 Design for Flexure - Constructibility Check - Negative Moment Region

The girder must also be checked for flexure during construction **LRFD [6.10.3.2]**. The girder has already been checked in its final condition when it behaves as a composite section. The constructibility must also be checked for the girder prior to the hardening of the concrete deck when the girder behaves as a noncomposite section.

For discretely braced flanges in compression with a compact or noncompact web and with f_l equal to zero (interior girder), equation 2 is used. This check is similar to the check performed in E24-1.20 and will not be checked here.

For the interior girder in this case (where $f_l = 0$), the sizes of the flanges at the pier section are controlled by the strength limit state flexural resistance checks illustrated previously. Therefore, separate constructibility checks on the flanges need not be made. However, the web bend buckling resistance of the noncomposite pier section during construction must be checked according to equation 3 of **LRFD [6.10.3.2.1]**, as follows:

$$f_{bu} \le \phi_f \cdot F_{crw}$$

Check first if the noncomposite section at the pier is a nonslender web section. From Table E24-1.3-3 **LRFD [6.10.6.2.3]**:

$D_c := 28.718 - 2.75$ $\boxed{D_c = 25.97}$ in

$$\boxed{\frac{2 \cdot D_c}{t_w} = 103.87}$$

$$\boxed{\lambda_{rw} = 137.27}$$

$$\frac{2 \cdot D_c}{t_w} < \lambda_{rw}$$ OK

The section is therefore a nonslender web section (i.e. a noncompact web section), web bend buckling need not be checked in this case according to **LRFD [6.10.3.2.1]**.

In addition to checking the flexural resistance during construction, the shear resistance in the web must also be checked prevent shear buckling of the web during construction as follows **LRFD [6.10.3.3]**:

$V_{cr} := C \cdot V_p$ $\boxed{V_{cr} = 367.53}$ kips

$V_r := \phi_v \cdot V_{cr}$ $\boxed{V_r = 367.53}$ kips

$V_u := (1.25 \cdot 111.5)$ $\boxed{V_u = 139.38}$ kips OK

Therefore, the design section at the pier satisfies the constructibility specification checks.

E24-1.26 Check Wind Effects on Girder Flanges - Negative Moment Region

Wind effects generally do not control a steel girder design, and they are generally considered for the exterior girders only **LRFD [C6.10.1.6 & C4.6.2.7.1]**. However, for illustrative purposes, wind effects are presented below for the girder design section at the pier. A bridge height of greater than 30 feet is used in this design step to illustrate the required computations **LRFD [3.8.1.1]**.

The stresses in the bottom flange are combined as follows **LRFD [6.10.8.1.1]**:

$$\left(f_{bu} + \frac{1}{3} f_l\right) \le \phi_f \cdot F_{nc}$$

$$f_l := \frac{6 \cdot M_w}{t_{fb} \cdot b_{fb}^{\ 2}}$$ (LRFD 6.10.1.6)

Since the deck provides horizontal diaphragm action and since there is wind bracing in the superstructure, the maximum wind moment, M_w, on the loaded flange is determined as follows:

$$M_w := \frac{W \cdot L_b^{\ 2}}{10}$$

$$\frac{L_b}{12} = 20.00$$ ft

$$W := \frac{\eta \cdot \gamma \cdot P_D \cdot d}{2}$$

$\eta := 1.0$

$\gamma := 0.40$ for Strength V Limit State

Assume that the bridge is to be constructed in a city. The design horizontal wind pressure, P_D, is computed as follows **LRFD [3.8.1.2]**:

$$P_D := P_B \cdot \left(\frac{V_{DZ}}{V_B}\right)^2$$

Where:

P_B = Base wind pressure **LRFD [Table 3.8.1.1-1]** (ksf)

V_{DZ} = Design wind velocity at design elevation Z (mph)

V_B = Base wind velocity of 100 mph for a 30.0 ft height

$P_B := 0.050$ ksf

$V_B := 100$ mph

$$V_{DZ} := 2.5 \cdot V_o \cdot \left(\frac{V_{30}}{V_B}\right) \cdot \ln\left(\frac{Z}{Z_o}\right)$$

Where:

V_{30} = Wind velocity at 30.0 feet above low ground or above design water level (mph)

V_0 = Friction velocity **LRFD [Table 3.8.1.1-1]** (mph)

Z = Height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 30.0 feet

Z_0 = Friction length of upstream fetch **LRFD [Table 3.8.1.1-1]** (ft)

$V_o := 12.0$ MPH for a bridge located in a city

$V_{30} := 60$ MPH assumed wind velocity at 30 feet above low ground or above design water level at bridge site

$V_B = 100$ MPH

$Z := 35$ ft assumed height of structure at which wind loads are being calculated as measured from low ground or from water level

$Z_o := 8.20$ ft for a bridge located in a city

$$V_{DZ} := 2.5 \cdot V_o \cdot \left(\frac{V_{30}}{V_B}\right) \cdot \ln\left(\frac{Z}{Z_o}\right)$$ $\boxed{V_{DZ} = 26.12}$ MPH

$$P_D := P_B \cdot \left(\frac{V_{DZ}}{V_B}\right)^2$$ $\boxed{P_D = 0.0034}$ ksf

$d := 8.45$ ft from bottom of girder to top of barrier

$W := P_D \cdot d$ $\boxed{W = 0.0288}$ kips/ft

LRFD [3.8.1.2.1] states that the total wind loading, W, must not be taken less than 0.30 klf on beam or girder spans, therefore use P_D as computed below:

$W := 0.30$ kips/ft

$$P_D := \frac{W}{d}$$ $\boxed{P_D = 0.0355}$ ksf

After the design horizontal wind pressure has been computed, the factored wind force per unit length applied to the flange is computed as follows **LRFD [C4.6.2.7.1]**:

$$W := \frac{\eta \cdot \gamma \cdot P_D \cdot d}{2}$$ $\boxed{W = 0.060}$ kips/ft

Next, the maximum lateral moment in the flange due to the factored wind loading is computed as follows:

$$M_w := \frac{W \cdot \left(\frac{L_b}{12}\right)^2}{10}$$ $\boxed{M_w = 2.40}$ kip-ft

Finally, the flexural stress at the edges of the bottom flange due to factored wind loading is computed as follows **LRFD [6.10.8.1.1]**:

$t_{fb} := 2.75$ in

$b_{fb} := 14.0$ in

$$f_l := \frac{6 \cdot -M_w \cdot 12}{t_{fb} \cdot b_{fb}^2}$$ $\boxed{f_l = -0.321}$ ksi

The load factor for live load is 1.35 for the Strength V Limit State. However, it is 1.75 for the Strength I Limit State, which we have already investigated. Therefore, it is clear that wind effects will not control the design of this steel girder. Nevertheless, the following computations are presented simply to demonstrate that wind effects do not control this design:

$$f_{bu} := [(1.25 \cdot -16.56) + (1.25 \cdot -2.05)] + [(1.50 \cdot -1.94) + (1.35 \cdot -12.26)]$$

$\boxed{f_{bu} = -42.72}$ ksi

$\boxed{f_{bu} + \frac{1}{3} f_l = -42.83}$ ksi

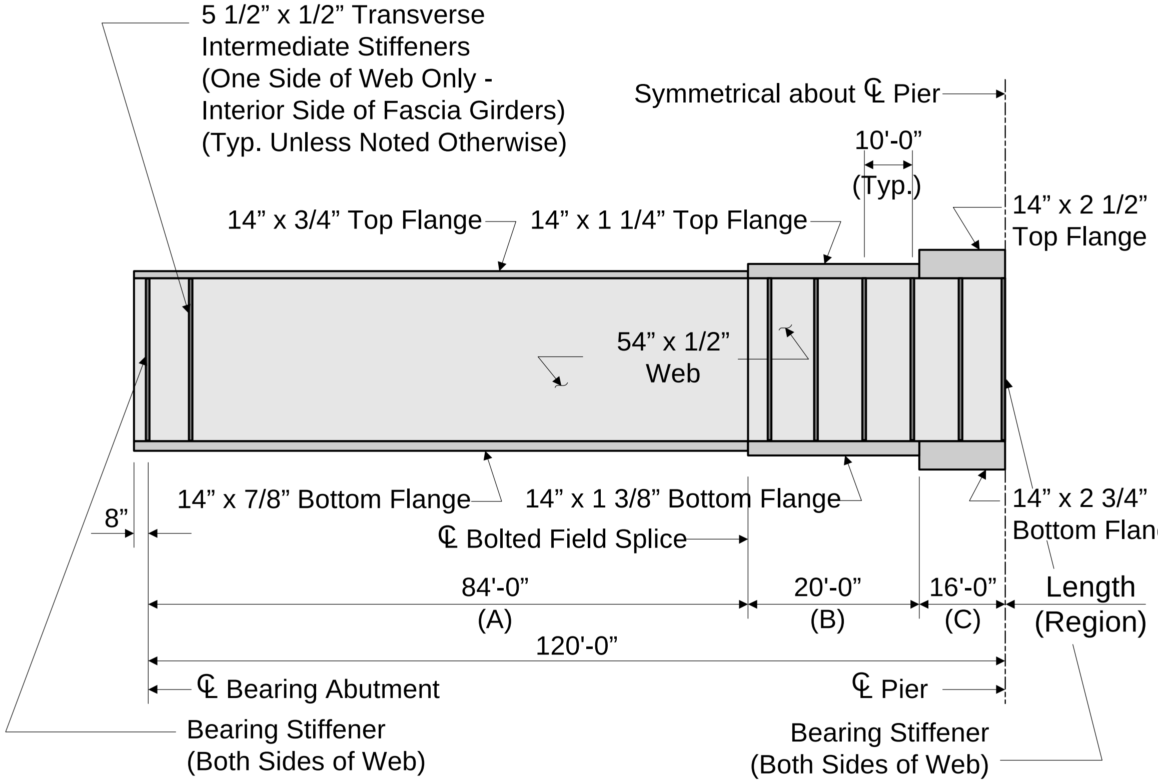
$\boxed{F_{nc} = 50.00}$ ksi

$f_{bu} + \frac{1}{3} f_l \le \phi_f \cdot F_{nc}$ OK

E24-1.27 Draw Schematic of Final Steel Girder Design

Since all of the specification checks were satisfied, the trial girder section presented in E24-1.2 is acceptable. If any of the specification checks were not satisfied or if the design were found to be overly conservative, then the trial girder section would need to be revised appropriately, and the specification checks would need to be repeated for the new trial girder section.

The following is a schematic of the final steel girder configuration:



<u>Figure E24-1.27-1</u>
Final Plate Girder Elevation

For this design example, only the location of maximum positive moment, the location of maximum negative moment, and the location of maximum shear were investigated. However, the above schematic shows the plate sizes and stiffener spacing throughout the entire length of the girder.

Design computations for shear connectors and bearing stiffeners now follow.

E24-1.28 Design Shear Connectors

For continuous composite bridges, shear connectors are normally provided throughout the length of the bridge. In the negative flexure region, since the longitudinal reinforcement is considered to be a part of the composite section, shear connectors must be provided **LRFD [6.10.10.1]**.

Studs are used as shear connectors. The shear connectors must permit a thorough compaction of the concrete to ensure that their entire surfaces are in contact with the concrete. In addition, the shear connectors must be capable of resisting both horizontal and vertical movement between the concrete and the steel.

The following figure shows the stud shear connector proportions, as well as the location of the stud head within the concrete deck.

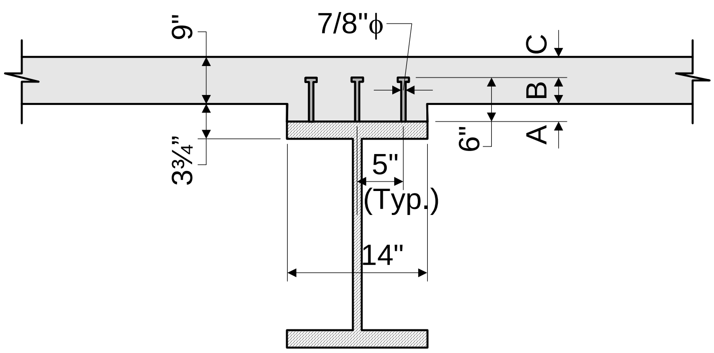


Figure E24-1.28-1
Stud Shear Connectors

Shear Connector Embedment			
Flexure Region	**A**	**B**	**C**
Positive	3.00"	3.00"	6.00"
Intermediate	2.50"	3.50"	5.50"
Negative	1.25"	4.75"	4.25"

Table E24-1.28-1
Shear Connector Embedment

The ratio of the height to the diameter of a stud shear connector must not be less than 4.0 **LRFD [6.10.10.1.1]**. For this design example, the ratio is computed based on the dimensions presented in Figure E24-1.28-1, as follows:

$Height_{stud} := 6.0$ in

$Diameter_{stud} := 0.875$ in

$$\frac{Height_{stud}}{Diameter_{stud}} = 6.86$$ OK

The pitch of the shear connectors must be determined to satisfy the fatigue limit state as specified in **LRFD [6.10.10.2 & 6.10.10.3]**, as applicable. The resulting number of shear connectors must not be less than the number required to satisfy the strength limit states as specified in **LRFD [6.10.10.4]**.

The pitch, p, of the shear connectors must satisfy the following equation **LRFD [6.10.10.1.2]**:

$$p \le \frac{n \cdot Z_r}{V_{sr}}$$

Where:

n = Number of shear connectors in a cross-section

Z_r = Shear fatigue resistance of an individual shear connector **LRFD [6.10.10.2]** (kip)

V_{sr} = Horizontal fatigue shear range per unit length (kip-in)

The shear fatigue resistance of an individual shear connector, Z_r, is taken as:

$ADTT_{SL} = 3000$ > 960, Therefore, use Fatigue 1 load combinations with fatigue shear resistance for infinite life as follows:

$$Z_r := 5.5 \cdot d^2$$

Where:

d = Diameter of the stud (in)

The horizontal fatigue shear range per unit length, V_{sr}, is taken as:

$$V_{sr} := \sqrt{V_{fat}^2 + F_{fat}^2}$$

Where:

V_{fat} = Longitudinal fatigue shear range per unit length

F_{fat} = Radial fatigue shear range per unit length (kip-in)

The longitudinal fatigue shear range per unit length, V_{fat}, is taken as:

$$V_{fat} := \frac{V_f \cdot Q}{I}$$

Where:

V_f = Vertical shear force range under the fatigue load combination in **LRFD [Table 3.4.1-1]** with the fatigue live load taken as specified in **LRFD [3.6.1.4]** (kip)

Q = First moment of the transformed short-term area of the concrete deck about the neutral axis of the short-term composite section (in^3)

I = Moment of inertia of the short-term composite section (in^4)

The radial fatigue shear range per unit length, F_{fat}, is taken as the larger of:

$$F_{fat1} := \frac{A_{bot} \cdot \sigma_{flg} \cdot l}{w \cdot R}$$

$$F_{fat2} := \frac{F_{rc}}{w}$$

Where:

A_{bot} = Area of the bottom flange (in^2)

σ_{flg} = Range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending (ksi)

l = Distance between brace points (ft)

w = Effective length of deck (in) taken as 48.0 in, except at end supports where w may be taken as 24.0 in

R = Minimum girder radius within the panel (ft)

F_{rc} = Net range of cross-frame or diaphragm force at the top flange (kip)

Since this bridge utilizes straight spans and has no skew, the radial fatigue shear range, F_{fat} is taken as zero. Therefore:

$$V_{sr} := V_{fat}$$

In the positive flexure region, the maximum fatigue live load shear range is located at the abutment. For illustration purposes, this example uses the average fatigue live load shear range in the positive moment region and assumes it acts at 0.4L. In reality, the required pitch should be calculated throughout the entire length of the girder. The actual pitch should be chosen such that it is less than or equal to the required pitch. The factored average value is computed as follows:

$V_f := 1.5 \cdot (44.5)$ $\boxed{V_f = 66.75}$ kips

The parameters I and Q are based on the short-term composite section and are determined using the deck within the effective flange width. In the positive flexure region:

$n := 3$ (see Figure E24-1.28-1)

$I := 70696.16$ in^4 (see Table E24-1.3-1)

$Q := \left[\frac{(8.5) \cdot (120)}{8}\right] \cdot (62.875 - 52.777)$ $\boxed{Q = 1287.49}$ in^3

$V_{fat} := \frac{V_f \cdot Q}{I}$ $\boxed{V_{fat} = 1.22}$ kip/in

$V_{sr} := V_{fat}$ $\boxed{V_{sr} = 1.22}$ kip/in

$d := 0.875$ in

$Z_r := 5.5 \cdot d^2$ $\boxed{Z_r = 4.21}$ kips

$p := \dfrac{n \cdot Z_r}{V_{sr}}$ $\boxed{p = 10.39}$ in

In the negative flexure region:

$n := 3$ (see Figure E24-1.28-1)

From **LRFD [C6.10.10.1.2]**, in the negative flexure region, the parameters I and Q may be determined using the reinforcement within the effective flange width for negative moment, unless the concrete slab is considered to be fully effective for negative moment in computing the longitudinal range of stress, as permitted in **LRFD [6.6.1.2.1]**. For this design example, I and Q are assumed to be computed considering the concrete slab to be fully effective.

$I := 139158.7$ in^4 (see Table E24-1.3-3)

$Q := \left[\dfrac{(8.5) \cdot (120)}{8}\right] \cdot (64.750 - 48.868)$ $\boxed{Q = 2024.95}$ in^3

$V_f := 1.5 \cdot (47.4)$ $\boxed{V_f = 71.10}$ kips

$V_{fat} := \dfrac{V_f \cdot Q}{I}$ $\boxed{V_{fat} = 1.03}$ kip/in

$V_{sr} := V_{fat}$ $\boxed{V_{sr} = 1.03}$ kip/in

$p := \dfrac{n \cdot Z_r}{V_{sr}}$ $\boxed{p = 12.21}$ in

Therefore, based on the above pitch computations to satisfy the fatigue limit state, use the following pitch throughout the entire girder length:

$p := 10$ in

As stated earlier, the shear connector pitch typically is not the same throughout the entire length of the girder. In reality, most girder designs use a variable pitch, which is beneficial economically.

However, for simplicity in this design example, a constant shear connector pitch of 10 inches will be used.

In addition, the shear connectors must satisfy the following pitch requirements **LRFD [6.10.10.1.2]**:

$p \leq 24$ in OK

$p \geq 6 \cdot d$ $\boxed{6 \cdot d = 5.25}$ in OK

For transverse spacing, the shear connectors must be placed transversely across the top flange of the steel section and may be spaced at regular or variable intervals **LRFD [6.10.10.1.3]**.

Stud shear connectors must not be closer than 4.0 stud diameters center-to-center transverse to the longitudinal axis of the supporting member.

$4 \cdot d = 3.50$ in

$Spacing_{transverse} := 5.0$ in (see Figure E24-1.28-1) OK

In addition, the clear distance between the edge of the top flange and the edge of the nearest shear connector must not be less than 1.0 inch.

$D_{clear} := \frac{14}{2} - 5 - \frac{d}{2}$ $\boxed{D_{clear} = 1.56}$ in OK

The clear depth of concrete cover over the tops of the shear connectors should not be less than 2.0 inches, and shear connectors should penetrate at least 2.0 inches into the deck **LRFD [6.10.10.1.4]**. Based on the shear connector penetration information presented in Table E24-1.28-1, both of these requirements are satisfied.

For the strength limit state, the factored resistance of the shear connectors, Q_r, is computed as follows **LRFD [6.10.10.4.1]**:

$Q_r := \phi_{sc} \cdot Q_n$

$\phi_{sc} := 0.85$ (LRFD 6.5.4.2)

The nominal shear resistance of one stud shear connector embedded in a concrete slab, Q_n, is computed as follows **LRFD [6.10.10.4.3]**:

$Q_n := 0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c} \leq A_{sc} \cdot F_u$

Where:

A_{sc} = Cross-sectional area of a stud shear connector (in^2)

F_u = Specified minimum tensile strength of a stud shear connector from **LRFD [6.4.4]** (ksi)

$A_{sc} := \pi \cdot \frac{d^2}{4}$ $\boxed{A_{sc} = 0.601}$ in^2

$F_u := 60.0$ ksi

$E_c := 3834$ ksi

$Q_n := \min\left(0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c}, A_{sc} \cdot F_u\right)$ $\boxed{Q_n = 36.08}$ kips

$Q_r := \phi_{sc} \cdot Q_n$ $\boxed{Q_r = 30.67}$ kips

The number of shear connectors provided over the section being investigated must not be less than the following **LRFD [6.10.10.4.1]**:

$$n := \frac{P}{Q_r}$$

For continuous spans that are composite for negative flexure in their final condition, the nominal shear force, P, must be calculated for the following regions **LRFD [6.10.10.4.2]**:

1. Between points of maximum positive design live load plus impact moments and adjacent ends of the member

2. Between points of maximum positive design live load plus impact moment and centerlines of adjacent interior supports

For Region 1:

$$P := \sqrt{P_p^{\,2} + F_p^{\,2}}$$

Where:

P_p = Total longitudinal shear force in the concrete deck at the point of maximum positive live load plus impact moment (kips)

F_p = Total radial shear force in the concrete deck at the point of maximum positive live load plus impact moment (kips)

The total longitudinal shear force in the concrete deck at the point of maximum positive live load plus impact moment, P_p, is taken as the lesser of:

$$P_{1p} := 0.85 \cdot f'_c \cdot b_s \cdot t_s$$

or

$$P_{2p} := F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}$$

$t_{ft} := 0.875$ in (see E24-1.27)

$t_{fc} := 0.75$ in (see E24-1.27)

$$P_p := \min\left(0.85 \cdot f'_c \cdot b_s \cdot t_s, F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}\right)$$

$\boxed{P_p = 2488}$ kips

For straight spans or segments, F_p may be taken equal to zero which gives **LRFD [6.10.10.4.2]**:

$P := P_p$ $\boxed{P = 2488}$ kips

Therefore, the number of shear connectors provided between the section of maximum positive moment and each adjacent end of the member must not be less than the following **LRFD [6.10.10.4.1]**:

$n := \frac{P}{Q_r}$ $\boxed{n = 81.1}$

For region 2:

$$P := \sqrt{P_T^{\,2} + F_T^{\,2}}$$

Where:

P_T = Total longitudinal shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kips)

F_T = Total radial shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kips)

The total longitudinal shear force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support, P_T, is taken as:

$$P_T := P_p + P_n$$

Where:

P_n = Total longitudinal shear force in the concrete deck over an interior support (kips)

The total longitudinal shear force in the concrete deck over an interior support, P_n, is taken as the lesser of:

$$P_{1n} := F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}$$

or

$$P_{2n} := 0.45 \cdot f'_c \cdot b_s \cdot t_s$$

$t_{ft} := 2.5$ in (see E24-1.27)

$t_{fc} := 2.75$ in (see E24-1.27)

$$P_n := \min\left(F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}, 0.45 \cdot f'_c \cdot b_s \cdot t_s\right)$$

$\boxed{P_n = 1836}$ kips

$P_T := P_p + P_n$ $\boxed{P_T = 4324}$ kips

For straight spans or segments, F_T may be taken equal to zero which gives:

$P := P_T$ $\boxed{P = 4324}$ kips

Therefore, the number of shear connectors provided between the section of maximum positive moment and the centerline of the adjacent interior pier must not be less than the following **LRFD [6.10.10.4.1]**:

$$n := \frac{P}{Q_r} \qquad \boxed{n = 141.0}$$

The distance between the end of the girder and the location of maximum positive moment is approximately equal to:

$L := 48.0$ ft (see Table E24-1.4-2)

Using a pitch of 10 inches, as previously computed for the fatigue limit state, and using the above length, the number of shear connectors provided is as follows:

$$n := 3 \cdot \frac{L \cdot (12)}{p} \qquad \boxed{n = 172.8} \qquad \text{OK}$$

Similarly the distance between the section of the maximum positive moment and the interior support is equal to:

$L := 120.0 - 48.0$ $\boxed{L = 72.0}$ ft (see Table E24-1.4-2)

Using a pitch of 10 inches, as previously computed for the fatigue limit state, and using the above length, the number of shear connectors provided is as follows:

$$n := 3 \cdot \frac{L \cdot (12)}{p} \qquad \boxed{n = 259.2} \qquad \text{OK}$$

Therefore, using a pitch of 10 inches for each row, with three stud shear connectors per row, throughout the entire length of the girder satisfies both the fatigue limit state requirements of **LRFD [6.10.10.1.2 & 6.10.10.2]** and the strength limit state requirements of **LRFD [6.10.10.4]**.

Use a shear stud spacing as illustrated in the following figure.

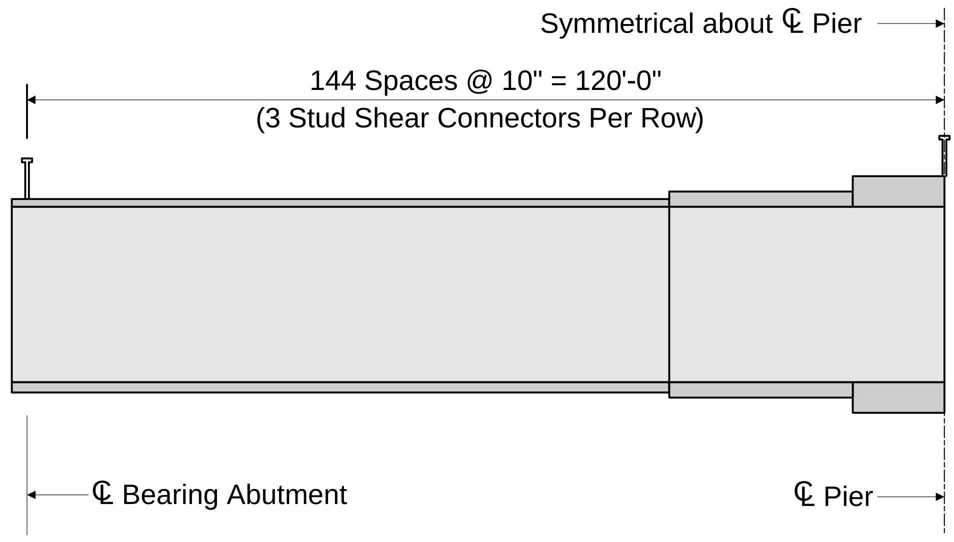


Figure E24-1.28-2
Shear Connector Spacing

E24-1.29 Design Bearing Stiffeners

Bearing stiffeners are required to resist the bearing reactions and other concentrated loads, either in the final state or during construction **LRFD [6.10.11.2.1]**.

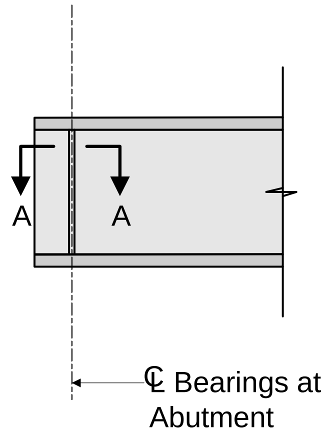
For plate girders, bearing stiffeners are required to be placed on the webs at all bearing locations. At all locations supporting concentrated loads where the loads are not transmitted through a deck or deck system, either bearing stiffeners are to be provided or the web must satisfy the provisions of **LRFD [Appendix D6.5]**.

Therefore, for this design example, bearing stiffeners are required at both abutments and at the pier. The following design of the abutment bearing stiffeners illustrates the bearing stiffener design procedure.

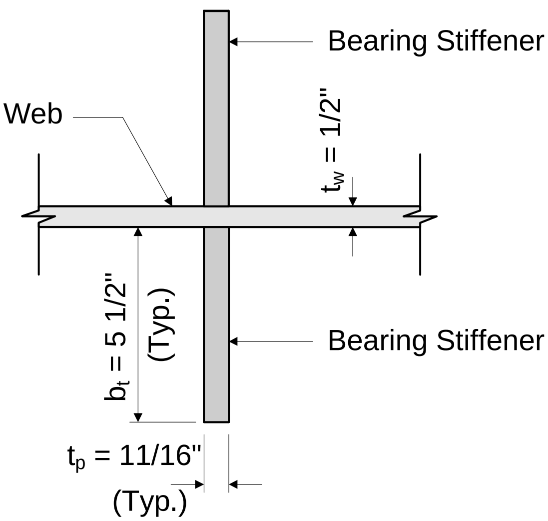
The bearing stiffeners in this design example consist of one plate welded to each side of the web. The connections to the web will be designed to transmit the full bearing force due to factored loads and is presented in E24-1.30.

The stiffeners extend the full depth of the web and, as closely as practical, to the outer edges of the flanges.

The following figure illustrates the bearing stiffener layout at the abutments.



Partial Girder Elevation at Abutment



Section A-A

Figure E24-1.29-1
Bearing Stiffeners at Abutments

The projecting width, b_t, of each bearing stiffener element must satisfy the following equation **LRFD [6.10.11.2.2]**. This provision is intended to prevent local buckling of the bearing stiffener plates.

$$b_t \leq 0.48 \cdot t_p \cdot \sqrt{\frac{E}{F_{ys}}}$$

Where:

t_p = Tickness of the projecting stiffener element (in)

F_{ys} = Specified minimum yield strength of the stiffener (ksi)

$b_t := 5.5$ in (see Figure E24-1.29-1)

$t_p := \frac{11}{16}$ in (see Figure E24-1.29-1)

$F_{ys} := 50$

$\boxed{0.48 \cdot t_p \cdot \sqrt{\frac{E_s}{F_{ys}}} = 7.95}$ in OK

The bearing resistance must be sufficient to resist the factored reaction acting on the bearing stiffeners **LRFD [6.10.11.2.3]**. The factored bearing resistance, R_{sbr}, is computed as follows:

$R_{sbr} := \phi_b \cdot R_{sbn}$

$\phi_b := 1.00$ (LRFD 6.5.4.2)

$R_{sbn} := 1.4 \cdot A_{pn} \cdot F_{ys}$

Where:

A_{pn} = Area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange (in^2)

Part of the stiffener must be clipped to clear the web-to-flange weld. Thus the area of direct bearing is less than the gross area of the stiffener. The bearing area, A_{pn}, is taken as the area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange. This is illustrated in the following figure:

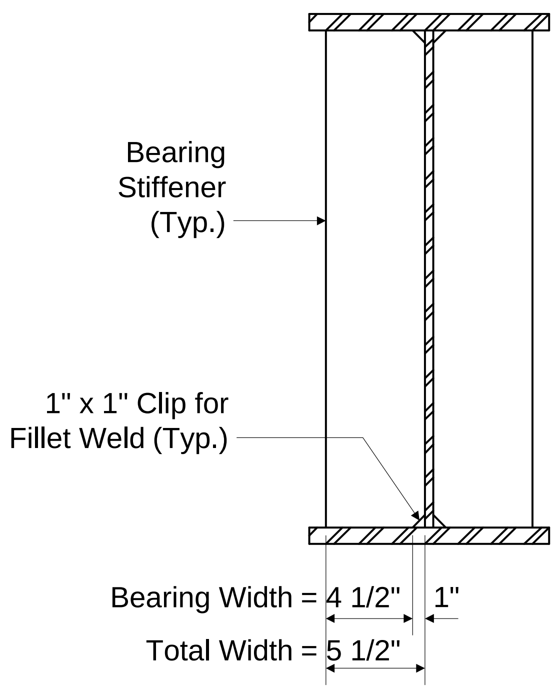


Figure E24-1.29-2
Bearing Width

$b_{brg} := b_t - 1.0$ $\boxed{b_{brg} = 4.50}$ in

$A_{pn} := 2b_{brg} \cdot t_p$ $\boxed{A_{pn} = 6.19}$ in^2

$R_{sbr} := \phi_b \cdot 1.4 \cdot A_{pn} \cdot F_{ys}$ $\boxed{R_{sbr} = 433.13}$ kips

The factored bearing reaction at the abutment is computed as follows, using load factors as presented in **LRFD [Table 3.4.1-1 & Table 3.4.1-2]** and using reactions obtained from Table E24-1.4-3 and Table E24-1.5-2:

$React_{Factored} := (1.25 \cdot 63.7) + (1.50 \cdot 7.4) + (1.75 \cdot 112.8)$

$\boxed{React_{Factored} = 288.13}$ kips

Therefore, the bearing stiffener at the abutment satisfies the bearing resistance requirements.

The final bearing stiffener check relates to the axial resistance of the bearing stiffeners **LRFD [6.10.11.2.4]**. The factored axial resistance is determined as specified in **LRFD [6.9.2.1]**. The radius of gyration is computed about the midthickness of the web, and the effective length is taken as 0.75D, where D is the web depth **LRFD [6.10.11.2.4a]**.

For stiffeners consisting of two plates welded to the web, the effective column section consists of the two stiffener elements, plus a centrally located strip of web extending not more than $9t_w$ on each side of the stiffeners **LRFD [6.10.11.2.4.b]**. This is illustrated in the following figure:

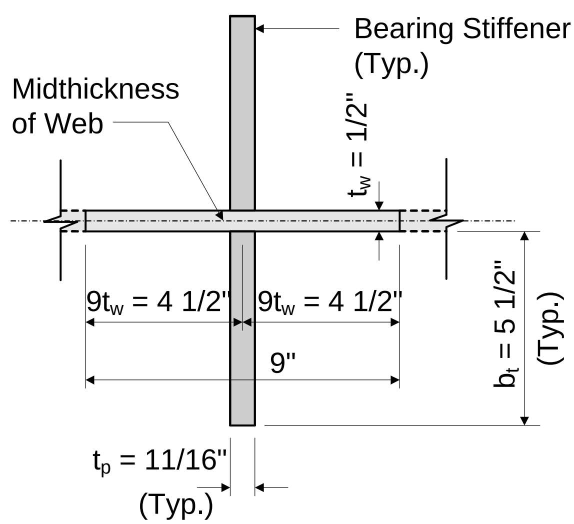


Figure E24-1.29-3
Bearing Stiffener Effective Section

$P_r := \phi_c \cdot P_n$ (LRFD 6.9.2.1)

$\phi_c := 0.90$ (LRFD 6.5.4.2)

Bearing stiffeners only need to be designed for Flexural Buckling failure (Torsional Buckling and Flexural Torsional Buckling are not applicable) **LRFD [6.9.4.1.1]**.

First, calculate the elastic critical buckling resistance, Pe, based on **LRFD [6.9.4.1.2]**.

$$P_e := \frac{A_g \cdot \left(\pi^2 \cdot E_s\right)}{\left(\frac{kl}{r_s}\right)^2}$$

Where:

kl	= Taken as 0.75D, where D is the web depth (in)
r_s	= Radius of gyration about the midthickness of the web (in)
A_g	= Cross-sectional area of the effective section (in^2)

$kl := (0.75) \cdot (54)$ $\boxed{kl = 40.50}$ in

$$I_s := \frac{\left(0.6875 \cdot 11.5^3\right) + \left(8.3125 \cdot 0.5^3\right)}{12}$$ $\boxed{I_s = 87.22}$ in^4

$A_g := (0.6875 \cdot 11.5) + (8.3125 \cdot 0.5)$ $\boxed{A_g = 12.06}$ in^2

$$r_s := \sqrt{\frac{I_s}{A_g}}$$ $\boxed{r_s = 2.69}$ in

$$P_e := \frac{A_g \cdot \left(\pi^2 \cdot E_s\right)}{\left(\frac{kl}{r_s}\right)^2}$$ $P_e = 15220$ kip

Next, calculate the equivalent nominal yield resistance, P_o, given as:

$P_o := Q \cdot F_y \cdot A_g$ (LRFD 6.9.4.1.1)

Where:

Q	= slender element reduction factor, taken as 1.0 for bearing stiffeners

$P_o := 1.0F_y \cdot A_g$ $P_o = 603$ kip

$$\frac{P_e}{P_o} = 25.23$$ Since $P_e/P_o > 0.44$, Use equation 1 from **LRFD [6.9.4.1.1]**.

$$P_n := \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o$$ $\boxed{P_n = 593.20}$ kips

$$P_r := \phi_c \cdot P_n$$ $\boxed{P_r = 533.88}$ kips

$\boxed{React_{Factored} = 288.13}$ kips OK

Therefore, the bearing stiffener at the abutment satisfies the axial bearing resistance requirements.

The bearing stiffener at the abutment satisfies all bearing stiffener requirements. Use the bearing stiffener as presented in Figure E24-1.29-2 and Figure E24-1.29-3.

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E24-2 Bolted Field Splice, LRFD

E24-2.1 Obtain Design Criteria

This splice design example shows design calculations conforming to the *AASHTO LRFD Bridge Design Specifications (Seventh Edition - 2015 Interims)* as supplemented by the *WisDOT Bridge Manual (January 2015)*.
Note: This example uses the girder from example E24-1.

Presented in Figure E24-2.1-1 is the steel girder configuration and the bolted field splice location. **LRFD [6.13.6.1.4a]** recommends locating splices near points of dead load contraflexure.

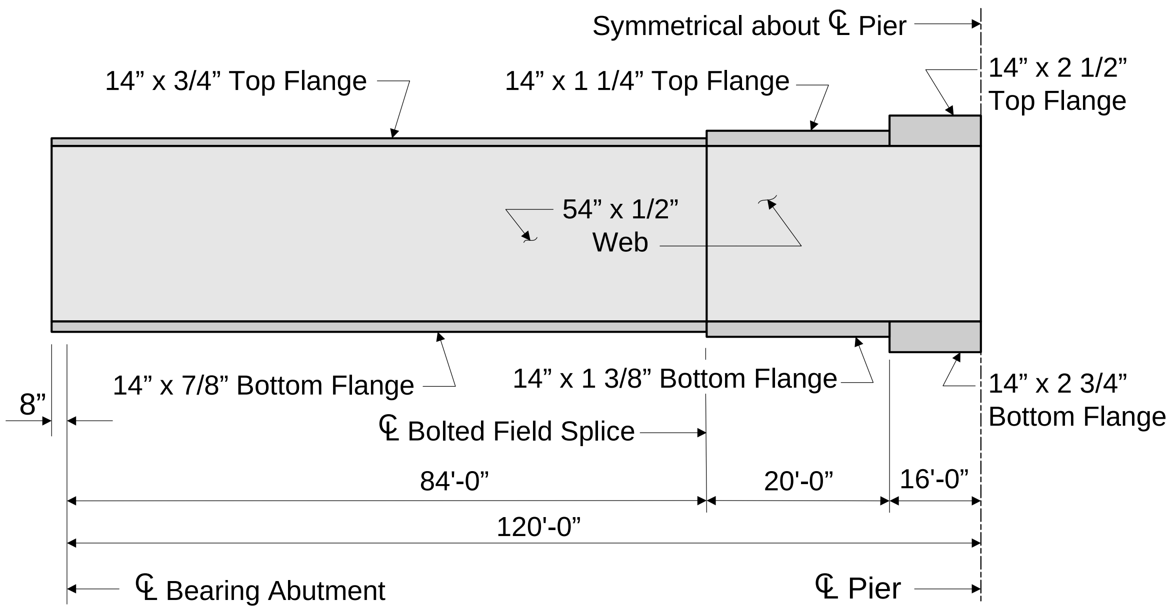


Figure E24-2.1-1
Plate Girder Elevation

The steel properties of the girder and splice plates are as follows:

Yield strength:

$F_y := 50$ ksi

Tensile strength:

$F_u := 65$ ksi

For specification checks requiring the flange yield strength:

$F_{yf} := 50$ ksi

The plate dimensions of the girder on the left side of the splice from Figure E24-2.1-1 are as follows:

Web thickness:

$t_w := 0.50$ in

Web depth:

$D := 54$ in

Top flange width:

$b_{fltL} := 14$ in

Top flange thickness:

$t_{fltL} := 0.75$ in

Bottom flange width:

$b_{flbL} := 14$ in

Bottom flange thickness:

$t_{flbL} := 0.875$ in

The plate dimensions of the girder on the right side of the splice from Figure E24-2.1-1 are as follows:

Web thickness:

$t_w := 0.50$ in

Web depth:

$D := 54$ in

Top flange width:

$b_{fltR} := 14$ in

Top flange thickness:

$t_{fltR} := 1.25$ in

Bottom flange width:

$b_{flbR} := 14$ in

Bottom flange thickness:

$t_{flbR} := 1.375$ in

The properties of the splice bolts are as follows:

Bolt diameter:

$d_{bolt} := 0.875$ in **LRFD [6.13.2.5]**

Bolt hole diameter (for design purposes add 1/16" to standard hole diameter):

$d_{hole} := 1.0$ in **LRFD [6.13.2.4.2-1]**

Bolt tensile strength:

$Fu_{bolt} := 120$ ksi **LRFD [6.4.3.1]**

The properties of the concrete deck are as follows:

Effective slab thickness:

$t_{seff} := 8.5$ in

Modular ratio:

$n := 8$

Haunch depth (measured from top of web):

$d_{haunch} := 3.75$ in

Effective flange width:

$W_{eff} := 120$ in

The area of longitudinal deck reinforcing steel in the negative moment region is for the top and bottom mat is given as number 6 bars at 7.5 inch spacing. The area of steel in the effective flange width is then:

For the top steel:

$$A_{deckreinftop} := (0.44) \cdot \frac{W_{eff}}{7.5} \qquad \boxed{A_{deckreinftop} = 7.04} \ \text{in}^2$$

For the bottom steel:

$$A_{deckreinfbot} := (0.44) \cdot \frac{W_{eff}}{7.5} \qquad \boxed{A_{deckreinfbot} = 7.04} \ \text{in}^2$$

Resistance factors **LRFD [6.5.4.2]**:

Flexure: $\phi_f := 1.0$

Shear: $\phi_v := 1.0$

Axial compression, composite: $\phi_c := 0.90$

Tension, fracture in net section: $\phi_u := 0.80$

Tension, yielding in gross section: $\phi_y := 0.95$

Bolts bearing on material: $\phi_{bb} := 0.80$

A325 and A490 bolts in shear: $\phi_s := 0.80$

Block shear: $\phi_{bs} := 0.80$

E24-2.2 Select Girder Section as Basis for Field Splice Design

Where a section changes at a splice, the smaller of the two connected sections shall be used in the design **LRFD [6.13.6.1.1]**. Therefore, the bolted field splice will be designed based on the left adjacent girder section properties. This will be referred to as the Left Girder throughout the calculations. The girder located to the right of the bolted field splice will be designated the Right Girder.

E24-2.3 Compute Flange Splice Design Loads

A summary of the unfactored moments at the splice from example 24-1 are listed below. The live loads include impact and distribution factors.

Dead load moments:

Noncomposite:

$M_{NDL} := -98.8$ kip-ft

Composite:

$M_{CDL} := 6.6$ kip-ft

Future wearing surface:

$M_{FWS} := 6.3$ kip-ft

Live load moments:

HL-93 positive:

$M_{PLL} := 1245.7$ kip-ft

HL-93 negative:

$M_{NLL} := -957.7$ kip-ft

Fatigue positive:

$M_{PFLL} := 375.6$ kip-ft

Fatigue negative:

$M_{NFLL} := -285.4$ kip-ft

Typically, splices are designed for the Strength I, Service II, and Fatigue I load combinations. The load factors for these load combinations are shown in Table E24.2.3-1 **LRFD [Tables 3.4.1-1 & 3.4.1-2]**:

	Load Factors					
	Strength I		Service II		Fatigue I	
Load	max	min	max	min	max	min
DC	1.25	0.90	1.00	1.00	-	-
DW	1.50	0.65	1.00	1.00	-	-
LL	1.75	1.75	1.30	1.30	1.50	1.50

Table E24-2.3-1
Load Factors

Flange stress computation procedure:

The stresses corresponding to the load combinations described above will be computed at the midthickness of the top and bottom flanges. The appropriate section properties and load factors for use in computing stresses are described below. Where necessary, refer to the signs of the previously documented design moments.

Strength I load combination:

The flexural stresses due to the factored loads at the strength limit state and for checking slip of the bolted connections at the point of splice shall be determined using the gross section properties **LRFD [6.13.6.1.4a]**.

Case 1: Dead load + positive live load

For this case, stresses will be computed using the gross section properties. The minimum load factor is used for the DC dead loads (noncomposite and composite) and the maximum load factor is used for the future wearing surface. The composite dead load and future wearing surface act on the 3n-composite slab section and the live load acts on the n-composite slab section **LRFD [6.10.1.1.1b]**.

Case 2: Dead load + negative live load

For this case, stresses will be computed using the gross section properties. The future wearing surface is excluded and the maximum load factor is used for the DC dead loads. The live load acts on the composite steel girder plus longitudinal reinforcement section. The composite dead load is applied to this section as well, as a conservative assumption for simplicity and convenience, since the net effect of the live load is to induce tension in the slab. **LRFD [6.10.1.1.1c]**.

Service II load combination:

Case 1: Dead load + positive live load

For this case, stresses will be computed using the gross steel section. The future wearing surface is included and acts, along with the composite dead load, on the 3n-composite slab section. The live load acts on the n-composite slab section.

Case 2: Dead load + negative live load

For this case, stresses will be computed using the gross steel section. The future wearing surface is excluded. The composite dead load acts on the 3n-composite slab section. The live load acts on the n-composite slab section.

Fatigue I load combination:

Case 1: Positive live load

For this case, stresses will be computed using the gross steel section. The live load acts on the n-composite slab section.

Case 2: Negative live load

For this case, stresses will be computed using the gross steel section. The live load acts on the n-composite slab section.

Section properties:

The effective flange area, A_e, is calculated from **LRFD [6.13.6.1.4c]**:

$$A_e := \left(\frac{\phi_u \cdot F_u}{\phi_y \cdot F_{yt}}\right) \cdot A_n \leq A_g$$

Where:

ϕ_u = Resistance factor for fracture of tension members **LRFD [6.5.4.2]**

ϕ_y = Resistance factor for yielding of tension members **LRFD [6.5.4.2]**

A_n = Net area of the tension flange (in^2) **LRFD [6.8.3]**

A_g = Gross area of the tension flange (in^2)

F_u = Specified minimum tensile strength of the tension flange (ksi) **LRFD [Table 6.4.1-1]**

F_{yt} = Specified minimum yield strength of the tension flange (ksi)

The gross area of the top and bottom flange of the steel girder is as follows:

$A_{gbot} := t_{flbL} \cdot b_{flbL}$ $\quad\quad$ $\boxed{A_{gbot} = 12.25}$ in^2

$A_{gtop} := t_{fltL} \cdot b_{fltL}$ $\quad\quad$ $\boxed{A_{gtop} = 10.50}$ in^2

The net area of the bottom flange of the steel girder is defined as the product of the thickness of the flange and the smallest net width **LRFD [6.8.3]**. The net width is determined by subtracting from the width of the flange the sum of the widths of all holes in the assumed failure chain, and then adding the quantity s^2 /4g for each space between consective holes in the chain. Since the bolt holes in the flanges are lined up transverse to the loading direction, the governing failure chain is straight across the flange (i.e., s^2 /4g is equal to zero).

The net area of the bottom and top flanges of the steel girder now follows:

$A_{nbot} := (b_{flbL} - 4 \cdot d_{hole}) \cdot t_{flbL}$ $\boxed{A_{nbot} = 8.75}$ in²

$A_{ntop} := (b_{fltL} - 4 \cdot d_{hole}) \cdot t_{fltL}$ $\boxed{A_{ntop} = 7.50}$ in²

With the gross and net areas identified, the effective tension area of the bottom and top flanges can now be computed as follows:

$F_{yt} := 50$ ksi

$A_{ebot} := \left(\frac{\phi_u \cdot F_u}{\phi_y \cdot F_{yt}}\right) \cdot A_{nbot}$ $\boxed{A_{ebot} = 9.58}$ in²

$A_{etop} := \left(\frac{\phi_u \cdot F_u}{\phi_y \cdot F_{yt}}\right) \cdot A_{ntop}$ $\boxed{A_{etop} = 8.21}$ in²

Check:

$A_{ebot} = 9.58$ < $A_{gbot} = 12.25$ OK

Effective bottom flange area:

$A_{ebot} = 9.58$ in²

Effective top flange area:

$A_{etop} = 8.21$ in²

The transformed effective area of the concrete flange of the steel girder is now determined as follows:

$$A_c = \frac{\text{Effective Slab Width}}{\text{Modular Ratio}} \times t_{seff}$$

$W_{eff} = 120.00$ in

$n = 8$ in

$t_{seff} = 8.50$ in

For the n-composite beam:

$A_c := \frac{W_{eff}}{n} \cdot t_{seff}$ $\boxed{A_c = 127.50}$ in²

For the 3n-composite beam:

$A_{c3n} := \frac{W_{eff}}{3n} \cdot t_{seff}$ $\boxed{A_{c3n} = 42.50}$ in²

The section properties for the Left Girder are calculated with the aid of Figure E24-2.3-1 shown below:

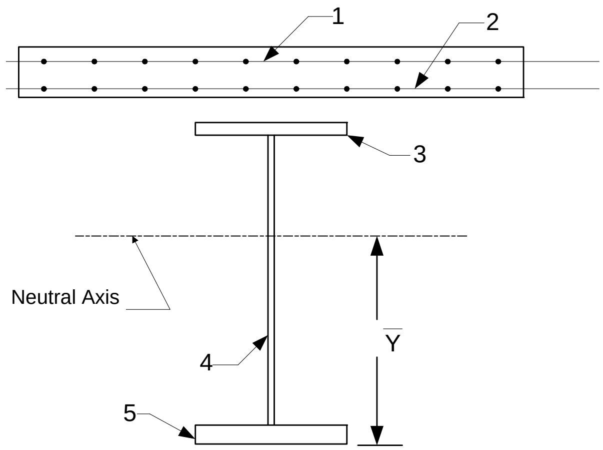


Figure E24-2.3-1
Girder, Slab and Longitudinal Reinforcement

The following tables contain the section properties for the left (i.e., smaller) girder section at the splice location. The properties in Table E24-2.3-2 are based on the gross area of the steel girder, and these properties are used for computation of stresses for the Service II and Fatigue Limit States. The properties in Tables E24-2.3-3 and E24-2.3-4 are based on the effective top flange and effective bottom flange of the steel girder, respectively, and these properties are used for computation of stresses for the Strength I Limit State.

Section Properties - Effective Top Flange Area						
Section	Area, A (Inches^2)	Centroid, d (Inches)	A*d (Inches^3)	I_o (Inches^4)	A*y^2 (Inches^4)	Itotal (Inches^4)
Girder only:						
Top flange	10.500	55.250	580.1	0.5	8441.1	8441.6
Web	27.000	27.875	752.6	6561.0	25.8	6586.8
Bottom flange	12.250	0.438	5.4	0.8	8576.1	8576.9
Total	49.750	26.897	1338.1	6562.3	17043.0	23605.3
Deck Steel:						
Girder	49.750	26.897	1338.1	23605.3	1066.7	24672.0
Top Steel	7.040	64.250	452.3	0.0	7538.3	7538.3
Total	56.790	31.527	1790.4	23605.3	8605.0	32210.3
Composite (3n):						
Girder	49.750	26.897	1338.1	23605.3	13668.5	37273.7
Slab	42.500	62.875	2672.2	255.9	16000.2	16256.0
Total	92.250	43.472	4010.3	23861.1	29668.6	53529.8
Composite (n):						
Girder	49.750	26.897	1338.1	23605.3	33321.4	56926.6
Slab	127.500	62.875	8016.6	767.7	13001.9	13769.5
Total	177.250	52.777	9354.7	24372.9	46323.2	70696.2
Section	y_{botmid} (Inches)	y_{topmid} (Inches)	S_{botweb} (Inches^3)	S_{botmid} (Inches^3)	S_{topmid} (Inches^3)	S_{topweb} (Inches^3)
Girder only	26.459	28.353	907.1	892.1	832.5	843.7
Deck Steel	31.090	23.723	1050.8	1036.0	1357.8	1379.6
Composite (3n)	43.035	11.778	1256.7	1243.9	4544.9	4694.4
Composite (n)	52.339	2.473	1362.1	1350.7	28583.9	33692.3

Table E24-2.3-2

Section Properties

Note: This example uses only the top layer of deck steel as it is WisDOT's policy to only include the top layer of bar steel in determining section properties and stresses.

Strength I Limit State stresses - Dead load + positive live load:

The section properties for this case have been calculated in Table E24-2.3-2. The stresses at the midthickness of the flanges are shown in Table E24-2.3-3, which immediately follows the sample calculation presented below.

A typical computation for the stresses occurring at the midthickness of the flanges is presented in the example below. The stress in the bottom flange of the girder is computed using the 3n-composite section for the composite dead load and future wearing surface, and the n-composite section for the live load:

$$f := \frac{M}{S}$$

Noncomposite DL:

Stress at the midthickness:

$f := f_{botgdr_1}$

Noncomposite DL Moment:

$M_{NDL} = -98.80$ kip-ft

Section modulus (girder only), from Table E24-2.3-2:

$S_{botgdr_1} := 892.1$ in^3

Stress due to the noncomposite dead load:

$$f_{botgdr_1} := \frac{M_{NDL} \cdot 12}{S_{botgdr_1}}$$ $\boxed{f_{botgdr_1} = -1.33}$ ksi

Composite DL:

Stress at the midthickness:

$f := f_{botgdr_2}$

Composite DL moment:

$M_{CDL} = 6.60$ kip-ft

Section modulus (3n-composite), from Table E24-2.3-2:

$S_{botgdr_2} := 1243.9$ in^3

Stress due to the composite dead load:

$$f_{botgdr_2} := \frac{M_{CDL} \cdot 12}{S_{botgdr_2}}$$ $\boxed{f_{botgdr_2} = 0.06}$ ksi

Future wearing surface:

Stress at the midthickness:

$f := f_{botgdr_3}$

FWS moment:

$M_{FWS} = 6.30$ kip-ft

Section modulus (3n-composite), From Table E24-2.3-2:

$S_{botgdr_3} := 1243.9$ in^3

Stress due to the composite dead load:

$$f_{botgdr_3} := \frac{M_{FWS} \cdot 12}{S_{botgdr_3}} \qquad \boxed{f_{botgdr_3} = 0.06} \text{ ksi}$$

Positive live load:

Stress at the midthickness:

$f := f_{botgdr_4}$

Live load moment:

$M_{PLL} = 1245.70$ kip-ft

Section modulus (n-composite), From Table E24-2.3-2:

$S_{botgdr_4} := 1350.7$ in^3

Stress due to the positive live load:

$$f_{botgdr_4} := \frac{M_{PLL} \cdot 12}{S_{botgdr_4}} \qquad \boxed{f_{botgdr_4} = 11.07} \text{ ksi}$$

The preceding stresses are now factored by their respective load factors to obtain the final factored stress at the midthickness of the bottom flange for this load case. The applicable load factors for this case were discussed previously **LRFD [Table 3.4.1-1 & 3.4.1-2]**.

$$f_{botgdr} := \left(0.90 \cdot f_{botgdr_1} + 0.90 \cdot f_{botgdr_2} + 1.50 \cdot f_{botgdr_3} + 1.75 \cdot f_{botgdr_4}\right)$$

$$\boxed{f_{botgdr} = 18.32} \text{ ksi}$$

The stresses at the midthickness of the top flange for this load case are computed in a similar manner. The section properties used to obtain the stresses in the top flange are also from Table E24-2.3-2.

The top and bottom flange midthickness stresses are summarized in Table E24-2.3-3, shown below.

Strength I - Dead Load + Positive Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.06	-0.02
FWS DL	6.30	0.06	-0.02
Live Load - HL-93	1245.70	11.07	-0.52
Summary of Factored Values			
Limit State			
Strength I	2106.45	18.32	0.33

Table E24-2.3-3

Strength I Flange Stresses for Dead + Pos. LL

The computation of the midthickness flange stresses for the remaining load cases are computed in a manner similar to what was shown in the sample calculation that preceded Table E24-2.3-3.

Strength I Limit State - Dead Load + Negative Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Strength I - Dead Load + Negative Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.08	-0.06
Live Load - HL-93	-957.70	-11.09	8.46
Summary of Factored Values			
Limit State			
Strength I	-1791.23	-20.98	16.52

Table E24-2.3-4

Strength I Flange Stresses for Dead + Neg. LL

Service II Limit State - Dead Load + Positive Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Service II - Dead Load + Positive Live Load			
Summary of Unfactored Values			
Loading	**Moment (K-ft)**	**f_{botmid} (ksi)**	**f_{topmid} (ksi)**
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.06	-0.02
FWS	6.30	0.06	-0.02
Live Load - HL-93	1245.70	11.07	-0.52
Summary of Factored Values			
Limit State			
Service II	1533.51	13.18	0.71

Table E24-2.3-5
Service II Flange Stresses for Dead + Pos. LL

Service II Limit State - Dead Load + Negative Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Service II - Dead Load + Negative Live Load			
Summary of Unfactored Values			
Loading	**Moment (K-ft)**	**f_{botmid} (ksi)**	**f_{topmid} (ksi)**
Noncomposite DL	-98.80	-1.33	1.42
Composite DL	6.60	0.06	0.00
Live Load - HL-93	-957.70	-8.51	0.40
Summary of Factored Values			
Limit State			
Service II	-1337.21	-12.33	1.94

Table E24-2.3-6
Service II Flange Stresses for Dead + Neg. LL

Fatigue Limit State - Positive Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Fatigue - Positive Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Live Load-Fatigue	375.60	3.34	-0.16
Summary of Factored Values			
Limit State			
Fatigue	563.40	5.01	-0.24

Table E24-2.3-7
Fatigue Flange Stresses for Positive LL

Fatigue Limit State - Negative Live Load:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Fatigue - Negative Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botmid} (ksi)	f_{topmid} (ksi)
Live Load-Fatigue	-285.40	-2.54	0.12
Summary of Factored Values			
Limit State			
Fatigue	-428.10	-3.80	0.18

Table E24-2.3-8
Fatigue Flange Stresses for Negative LL

Fatigue Limit State:

The computed stresses in the following table require the use of section properties from Table E24-2.3-2.

Fatigue - Live Load			
Summary of Unfactored Values			
Loading	Moment (K-ft)	f_{botweb} (ksi)	f_{topweb} (ksi)
Live Load-Pos	375.60	3.31	-0.13
Live Load-Neg	-285.40	-2.51	0.10
Summary of Factored Values			
Limit State			
Pos Fatigue	563.40	4.96	-0.20
Neg Fatigue	-428.10	-3.77	0.15

Table 42E2.3-9
Fatigue Web Stresses for Positive and Negative Live Load

A summary of the factored stresses at the midthickness of the top and bottom flanges for the Strength I, Service II, and Fatigue limit states are presented below in Tables E24-2.3-9 through E24-2.3-12. Table E24-2.3-12 also contains the top and bottom web fatigue stresses.

		Stress (ksi)	
Limit State	**Location**	**Dead + Pos. LL**	**Dead + Neg. LL**
Strength I	**Bottom Flange**	18.32	-20.98
	Top Flange	0.33	16.52

Table E24-2.3-10
Strength I Flange Stresses

		Stress (ksi)	
Limit State	**Location**	**Dead + Pos. LL**	**Dead + Neg. LL**
Service II	**Bottom Flange**	13.18	-12.33
	Top Flange	0.71	1.94

Table E24-2.3-11
Service II Flange Stresses

		Stress (ksi)	
Limit State	**Location**	**Positive LL**	**Negative LL**
Fatigue	**Bottom Flange**	5.01	-3.80
	Top Flange	-0.24	0.18
	Bottom of Web	4.96	-3.77
	Top of Web	-0.20	0.15

Table E24-2.3-12
Fatigue Flange and Web Stresses

Strength I minimum design force - controlling flange **LRFD [6.13.6.1.4c]**:

The next step is to determine the minimum design forces for the controlling flange of each load case (i.e., positive and negative live load). By inspection of Table E24-2.3-10, it is obvious that the bottom flange is the controlling flange for both positive and negative live load for the Strength I Limit State.

The minimum design force for the controlling flange, P_{cu}, is taken equal to the design stress, F_{cf}, times the smaller effective flange area, A_e, on either side of the splice. When a flange is in compression, the effective compression flange area shall be taken as $A_e = A_g$.

The calculation of the minimum design force is presented below for the load case of dead load with positive live load.

The minimum design stress for the controlling (bottom) flange, F_{cf}, is computed as follows **LRFD [6.13.6.1.4c]**:

$$F_{cf} := \frac{\left(\frac{f_{cf}}{R_h}\right) + \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g}{2} \geq 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g$$

Where:

f_{cf}	= Maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the point of splice (ksi)
R_g	= Flange resistance modification factor **LRFD [6.13.6.1.4c-3]**
R_h	= Hybrid factor **LRFD [6.10.1.10.1]**. For hybrid sections in which F_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0
α	= 1.0, except that a lower value equal to (F_n/F_{yf}) may be used for flanges where F_n is less than F_{yf}
ϕ_f	= Resistance factor for flexure **LRFD [6.5.4.2]**
F_n	= Nominal flexural resistance of the flange (ksi)
F_{yf}	= Specified minimum yield strength of the flange (ksi)

Maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the point of splice (from Table E24-2.3-10):

$f_{cf} := 18.32$ ksi

$R_h := 1.0$

$R_g := 1.0$

$\alpha := 1.0$

Resistance factor for flexure (see E24-2.1.1):

$\phi_f = 1.0$

$F_{yf} = 50.00$ ksi

$$F_{cf_1} := \frac{\left(\frac{f_{cf}}{R_h}\right) + \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g}{2} \qquad \boxed{F_{cf_1} = 34.16} \text{ ksi}$$

Compute the minimum required design stress:

$F_{cf_2} := 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g$ $\boxed{F_{cf_2} = 37.50}$ ksi

The minimum design stress for the bottom flange for this load case is:

$F_{cf} := \max\left(F_{cf_1}, F_{cf_2}\right)$ $\boxed{F_{cf} = 37.50}$ ksi

The minimum design force now follows:

$P_{cu} := F_{cf} \cdot A_e$

The gross area of the bottom flange is:

$A_{flbL} := b_{flbL} \cdot t_{flbL}$ $\boxed{A_{flbL} = 12.25}$ in^2

Since the bottom flange force for this load case is a tensile force, the effective area will be used. This value was computed previously to be:

$A_{ebot} = 9.58$ in^2

Therefore:

$P_{cu} := F_{cf} \cdot A_{ebot}$ $\boxed{P_{cu} = 359.21}$ kips

Table E24-2.3-13 presents the minimum design forces for the Strength I Limit State for both the positive and negative live load cases.

		Strength I Limit State			
		Controlling Flange			
Load Case	**Location**	**f_{cf} (ksi)**	**F_{cf} (ksi)**	**Area (in^2)**	**P_{cu} (kips)**
Dead + Pos. LL	**Bot. Flange**	18.32	37.50	9.58	359.21
Dead + Neg. LL	**Bot. Flange**	-20.98	37.50	12.25	459.38

Table E24-2.3-13
Controlling Flange Forces

In the above table, the design controlling flange force (P_{cu}) is a compressive force for negative live load.

Strength I minimum design force - noncontrolling flange **LRFD [6.13.6.1.4c]**:

The next step is to determine the minimum design forces for the noncontrolling flange of each load case (i.e., positive and negative live load). By inspection of Table 24E2.3-10, the top flange is the noncontrolling flange for both positive and negative live load for the Strength I Limit State.

The minimum design force for the noncontrolling flange, P_{ncu}, is taken equal to the design stress, F_{ncf}, times the smaller effective flange area, A_e, on either side of the splice. When a flange is in compression, the effective compression flange area shall be taken as $A_e = A_g$

The calculation of the minimum design force is presented below for the load case of dead load with positive live load.

The minimum design stress for the noncontrolling (top) flange, F_{ncf}, is computed as follows **LRFD [6.13.6.1.4c]**:

$$F_{ncf} := R_{cf} \cdot \left| \frac{f_{ncf}}{R_h} \right| \geq 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g$$

Where:

R_{cf} = The absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange

f_{ncf} = Flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with f_{cf} (ksi)

Maximum flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with fcf (see Table 24E2.3-10):

$f_{ncf} := 0.33$ ksi

Controlling flange design stress:

$F_{cf} = 37.50$ ksi

Controlling flange actual stress:

$f_{cf} = 18.32$ ksi

Controlling flange stress ratio:

$$R_{cf} := \left| \frac{F_{cf}}{f_{cf}} \right| \qquad \boxed{R_{cf} = 2.05}$$

$R_h = 1.00$

Therefore:

$$F_{ncf_1} := R_{cf} \cdot \left| \frac{f_{ncf}}{R_h} \right| \qquad \boxed{F_{ncf_1} = 0.68} \text{ ksi}$$

Compute the minimum required design stress:

$$F_{ncf_2} := 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf} \cdot R_g \qquad \boxed{F_{ncf_2} = 37.50} \text{ ksi}$$

The minimum design stress in the top flange is:

$$F_{ncf} := \max\left(F_{ncf_1}, F_{ncf_2}\right) \qquad \boxed{F_{ncf} = 37.50} \text{ ksi}$$

The minimum design force now follows:

$$P_{ncu} := F_{ncf} \cdot A_e$$

For the positive live load case, the top flange is in compression. The effective compression flange area shall be taken as:

$A_{etop} := A_{gtop}$ $\boxed{A_{etop} = 10.50}$ in^2

Therefore:

$P_{ncu} := F_{ncf} \cdot A_{etop}$ $\boxed{P_{ncu} = 393.75}$ kips (compression)

Table E24-2.3-14 presents the minimum design forces for the Strength I Limit State for both the positive and negative live load cases.

		Strength I Limit State			
		Noncontrolling Flange			
Load Case	**Location**	**f_{cf} (ksi)**	**F_{cf} (ksi)**	**Area (in^2)**	**P_{cu} (kips)**
Dead + Pos. LL	**Top Flange**	0.33	37.50	10.50	393.75
Dead + Neg. LL	**Top Flange**	16.52	37.50	8.21	307.89

Table E24-2.3-14
Noncontrolling Flange Forces

In the above table, the design noncontrolling flange force (P_{ncu}) is a compressive force for positive live load.

Service II Limit State flange forces **LRFD [6.13.6.1.4c]**:

Per the specifications, bolted connections for flange splices are to be designed as slip-critical connections for the service level flange design force. This design force shall be taken as the Service II design stress, F_s, multiplied by the smaller gross flange area on either side of the splice.

The Service II design stress, F_s, for the flange under consideration at a point of splice is defined as follows:

$$F_s := \frac{f_s}{R_h}$$

Where:

f_s = Maximum flexural Service II stress at the midthickness of the flange under consideration

The factored Service II design stresses and forces are shown in Table E24-2.3-15 below.

Load Case	Location	Service II Limit State F_s (ksi)	A_{gross} (in^2)	P_s (kips)
Dead + Pos. LL	Bot. Flange	13.18	12.25	161.49
	Top Flange	0.71	10.50	7.46
Dead + Neg. LL	Bot. Flange	-12.33	12.25	-151.06
	Top Flange	1.94	10.50	20.41

Table E24-2.3-15
Service II Flange Forces

It is important to note here that the flange slip resistance must exceed the larger of: (1) the Service II flange forces or (2) the factored flange forces from the moments at the splice due to constructibility (erection and/or deck pouring sequence) **LRFD [6.13.6.1.4a]**. However, in this design example, no special erection procedure is prescribed, therefore, the deck is assumed to be placed in a single pour. The constructibility moment is then equal to the noncomposite dead load moment shown at the beginning of this design step. By inspection, the Service II Limit State will control for checking of slip-critical connections for the flanges and the web in this example.

Fatigue Limit State stresses:

The final portion of this design step is to determine the range of the stresses at the midthickness of both flanges, and at the top and bottom of the web for the Fatigue Limit State. The ranges are calculated below and presented in Table E24-2.3-16.

A typical calculation of the stress range for the bottom flange is shown below.

From Tables E24-2.3-7 and E24-2.3-8, the factored stresses at the midthickness of the bottom flange are:

Case 1 - Positive Live Load:

$f_{spos} := 5.01$

Case 2 - Negative Live Load:

$f_{sneg} := -3.8$

The stress range is determined by:

$\Delta f := |f_{spos}| + |f_{sneg}|$ $\boxed{\Delta f = 8.81}$ ksi

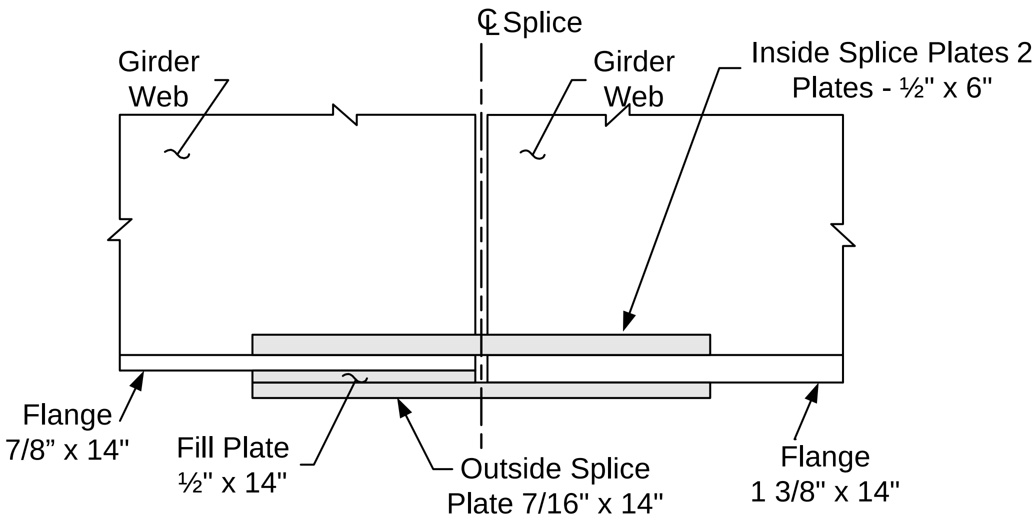
Location	Fatigue Limit State Stress Range (ksi)
Bottom Flange	8.81
Top Flange	0.42
Bottom of Web	8.73
Top of Web	0.35

Table E24-2.3-16
Fatigue Stress Ranges

E24-2.4 - Design Bottom Flange Splice

Splice plate dimensions:

The width of the outside plate should be at least as wide as the width of the narrowest flange at the splice. Therefore, try a 7/16" x 14" outside splice plate with two 1/2" x 6" inside splice plates. Include a 1/2" x 14" fill plate on the outside. Figure E24-2.4-1 illustrates the initial bottom flange splice configuration.



<u>Figure E24-2.4-1</u>
Bottom Flange Splice

The dimensions of the elements involved in the bottom flange splice from Figure E24-2.4-1 are:

Thickness of the inside splice plate:

$t_{in} := 0.50$ in

Width of the inside splice plate:

$b_{in} := 6$ in

Thickness of the outside splice plate:

$t_{out} := 0.4375$ in

Width of the outside splice plate:

$b_{out} := 14$ in

Thickness of the fill plate:

$t_{fill} := 0.50$ in

Width of the fill plate:

$b_{fill} := 14$ in

If the combined area of the inside splice plates is within ten percent of the area of the outside splice plate, then both the inside and outside splice plates may be designed for one-half the flange design force **LRFD [C6.13.6.4.1c]**.

Gross area of the inside and outside splice plates:

Inside:

$A_{gross\_in} := 2 \cdot t_{in} \cdot b_{in}$ $\boxed{A_{gross\_in} = 6.00}$ in^2

Outside:

$A_{gross\_out} := t_{out} \cdot b_{out}$ $\boxed{A_{gross\_out} = 6.13}$ in^2

Check:

$$\left(1 - \frac{A_{gross\_in}}{A_{gross\_out}}\right) \cdot 100\% = 2.04 \cdot \%$$

The combined areas are within ten percent.

If the areas of the inside and outside splice plates had differed by more than ten percent, the flange design force would be proportioned to the inside and outside splice plates. This is 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 inner and outer splice plates.

Yielding and fracture of splice plates:

Case 1 - Tension **LRFD [6.13.5.2]**:

At the Strength Limit State, the design force in the splice plates subjected to tension shall not exceed the factored resistances for yielding, fracture, and block shear.

From Table E24-2.3-13, the Strength I bottom flange tension design force is:

$P_{cu} = 359.21$ kips

The factored tensile resistance for yielding on the gross section, P_r, is taken from **LRFD [6.8.2.1]**:

$P_r := \phi_y \cdot P_{ny}$

Where:

P_{ny} = Nominal tensile resistance for yielding in gross section (kips)

= $F_y A_g$

F_y = Specified minimum yield strength (ksi)

A_g = Gross cross-sectional area of the member (in^2)

$P_r := \phi_y \cdot F_y \cdot A_g$

$F_y = 50.00$ ksi See E24-2.1

$\phi_y = 0.95$ See E24-2.1

For yielding of the outside splice plate:

$A_g := A_{gross\_out}$ $\boxed{A_g = 6.13}$ in^2

$P_r := \phi_y \cdot F_y \cdot A_g$ $\boxed{P_r = 290.94}$ kips

The outside splice plate takes half of the design load:

$P_r = 290.94$ > $\boxed{\frac{P_{cu}}{2} = 179.61}$ OK

For yielding of the inside splice plates:

$A_g := A_{gross\_in}$ $\boxed{A_g = 6.00}$ in^2

$P_r := \phi_y \cdot F_y \cdot A_g$ $\boxed{P_r = 285.00}$ kips

The inside splice plate takes half of the design load:

$P_r = 285.00$ kips > $\boxed{\frac{P_{cu}}{2} = 179.61}$ kips OK

The factored tensile resistance for fracture on the net section, P_r, is calculated by:

$P_r := \phi_u \cdot P_{nu}$

Where:

P_{nu} = Nominal tensile resistance for fracture in net section (kips)

= $F_u A_n R_p U$

F_u = Tensile strength (ksi)

R_p = Reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size.

A_n = Net area of the member (in^2) **LRFD [6.8.3]**

U = reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements, and as specified in **LRFD [6.8.2.2]** for other cases

$P_r := \phi_u \cdot F_u \cdot A_n \cdot R_p \cdot U$

$F_u = 65.00$ ksi See E24-2.1

$\phi_u = 0.80$ See E24-2.1

$U := 1.0$

$R_p := 1.0$

To compute the net area of the splice plates, assume four 7/8" bolts across the width of the splice plate.

The net width shall be determined for each chain of holes extending across the member along any transverse, diagonal or zigzag line. This is determined by subtracting from the width of the element the sum of the width of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain. For non-staggered holes, such as in this design example, the minimum net width is the width of the element minus the number of bolt holes in a line straight across the width **LRFD [6.8.3]**.

For fracture of the outside splice plate:

The net width is:

$d_{hole} = 1.00$ in (see E24-2.1)

$b_{n\_out} := b_{out} - 4 \cdot d_{hole}$ $\boxed{b_{n\_out} = 10.00}$ in

The nominal area is determined to be:

$A_{n\_out} := b_{n\_out} \cdot t_{out}$ $\boxed{A_{n\_out} = 4.38}$ in^2

The net area of the connecting element is limited to $0.85A_g$ **LRFD [6.13.5.2]**:

$A_n \le 0.85 \cdot A_g$

$A_{gross\_out} = 6.13$ in^2

$A_{n\_out} = 4.38$ in^2 < $\boxed{0.85 \cdot A_{gross\_out} = 5.21}$ in^2 OK

$P_r := \phi_u \cdot F_u \cdot A_{n\_out} \cdot U$ $\boxed{P_r = 227.50}$ kips

The outside splice plate takes half of the design flange force:

$P_r = 227.50$ kips > $\boxed{\dfrac{P_{cu}}{2} = 179.61}$ kips OK

For fracture of the inside splice plates:

The net width is:

$b_{n\_in} := b_{in} - 2 \cdot d_{hole}$ $\boxed{b_{n\_in} = 4.00}$ in

The nominal area is determined to be:

$A_{n\_in} := 2\left(b_{n\_in} \cdot t_{in}\right)$ $\boxed{A_{n\_in} = 4.00}$ in^2

The net area of the connecting element is limited to $0.85A_g$:

$A_n \le 0.85 \cdot A_g$

$A_{gross\_in} = 6.00$ in^2

$A_{n\_in} = 4.00$ in^2 < $\boxed{0.85 \cdot A_{gross\_in} = 5.10}$ in^2 OK

$P_r := \phi_u \cdot F_u \cdot A_{n\_in} \cdot U$ $\boxed{P_r = 208.00}$ kips

The inside splice plates take half of the design flange force:

$P_r = 208.00$ kips > $\boxed{\dfrac{P_{cu}}{2} = 179.61}$ kips OK

Case 2 - Compression **LRFD [6.13.6.1.4c]**:

From Table E24-2.3-13, the Strength I bottom flange compression design force is:

$P_{cu} := 459.38$ kips

This force is distributed equally to the inside and outside splice plates.

The factored resistance of the splice plate, R_r, is calculated from:

$R_r := \phi_c \cdot F_y \cdot A_s$

Where:

ϕ_c = resistance factor for compression **LRFD [6.5.4.2]**

F_y = Specified minimum yield strength of the splice plate (ksi)

A_s = Gross area of the splice plate (in^2)

$\phi_c = 0.90$ (see E24-2.1)

For yielding of the outside splice plate:

$A_s := A_{gross\_out}$ $\boxed{A_s = 6.13}$ in^2

$R_{r\_out} := \phi_c \cdot F_y \cdot A_s$ $\boxed{R_{r\_out} = 275.63}$ kips

$R_{r\_out} = 275.63$ kips > $\boxed{\frac{P_{cu}}{2} = 229.69}$ kips OK

For yielding of the inside splice plates:

$A_s := A_{gross\_in}$ $\boxed{A_s = 6.00}$ in^2

$R_{r\_in} := \phi_c \cdot F_y \cdot A_s$ $\boxed{R_{r\_in} = 270.00}$ kips

$R_{r\_in} = 270.00$ kips > $\boxed{\frac{P_{cu}}{2} = 229.69}$ kips OK

Block shear **LRFD [6.13.6.1.4c, 6.13.5.2 & 6.13.4]**:

All tension connections, including connection plates, splice plates and gusset plates, shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection. Block shear rupture will usually not govern the design of splice plates of typical proportion. However, the block shear checks are carried out here for completeness.

From Table E24-2.3-13, the Strength I bottom flange tension design force is:

$P_{cu} := 359.25$ kips

$$R_r := \phi_{bs} \cdot R_p \cdot \left(0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_u \cdot A_{tn}\right) \le \phi_{bs} \cdot R_p \cdot \left(0.58 \cdot F_y \cdot A_{vg} + U_{bs} \cdot F_u \cdot A_{tn}\right)$$

Where:

R_p = Reduction factor (1.0)

ϕ_{bs} = Resistance factor for block shear **LRFD [6.5.4.2]**

F_y = Specified minimum yield strength of the connected material (ksi)

A_{vg} = Gross area along the plane resisting shear stress (in^2)

F_u = Specified minimum tensile strength of the connected material (ksi) **LRFD [Table 6.4.1-1]**

A_{tn} = Net area along the plane resising tension stress (in^2)

A_{vn} = Net area along the plane resisting shear stress (in^2)

U_{bs} = Reduction factor for block shear rupture resistance taken equal to 0.50 when the tension stress is non-uniform and 1.0 when the tension stress is uniform

From E24-2.1:

$F_y = 50.00$ ksi

$F_u = 65.00$ ksi

$\phi_{bs} = 0.80$

Outside splice plate:

Failure mode 1:

A bolt pattern must be assumed prior to checking an assumed block shear failure mode. An initial bolt pattern for the bottom flange splice, along with the first assumed failure mode, is shown in Figure E24-2.4-2. The outside splice plate will now be checked for block shear.

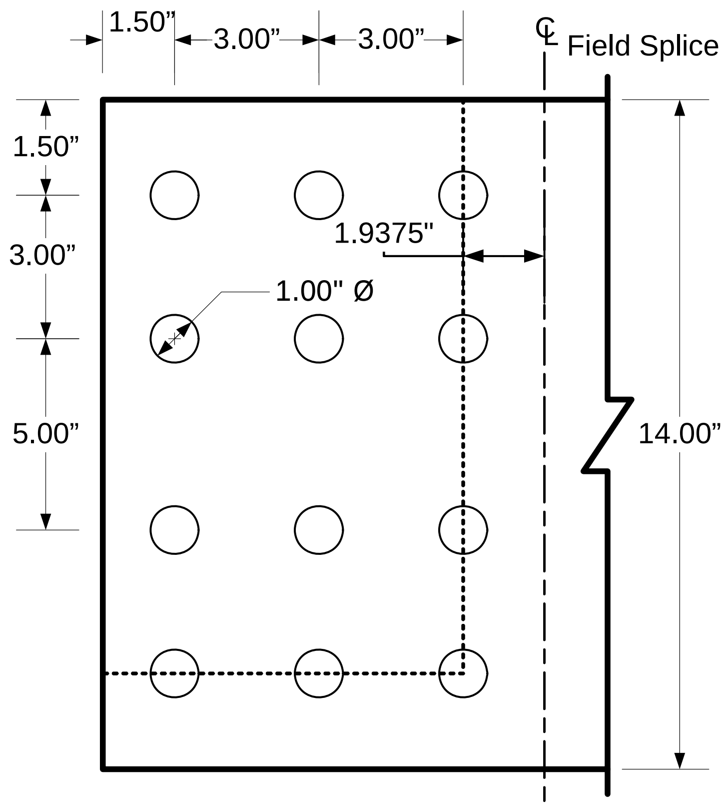


Figure E24-2.4-2
Outside Splice Plate - Failure Mode 1

Applying the factored resistance equations presented previously to the outside splice plate for failure mode 1:

Gross area along the plane resisting shear stress:

$A_{vg} := [2 \cdot (3.00) + 1.50] \cdot t_{out}$ $\boxed{A_{vg} = 3.28}$ in^2

Net area along the plane resisting shear stress:

$A_{vn} := \left[2 \cdot (3.00) + 1.50 - 2.5 \cdot d_{hole}\right] \cdot t_{out}$ $\boxed{A_{vn} = 2.19}$ in^2

Net area along the plane resisting tension stress:

$A_{tn} := \left[[2\cdot(3.00) + 5.00 + 1.50] - 3.5\cdot d_{hole}\right]\cdot t_{out}$ $\boxed{A_{tn} = 3.94}$ in^2

$U_{bs} := 1.0$

$R_{r1} := \phi_{bs}\cdot R_p\cdot\left(0.58\cdot F_u\cdot A_{vn} + U_{bs}\cdot F_u\cdot A_{tn}\right) = 270.73$

$R_{r2} := \phi_{bs}\cdot R_p\cdot\left(0.58\cdot F_y\cdot A_{vg} + U_{bs}\cdot F_u\cdot A_{tn}\right) = 280.88$

$R_r := \min\left(R_{r1}, R_{r2}\right)$

$\boxed{R_r = 270.73}$ kips

Check:

$R_r = 270.73$ kips > $\boxed{\frac{P_{cu}}{2} = 179.63}$ kips OK

Failure mode 2:

See Figure E24-2.4-3 for failure mode 2:

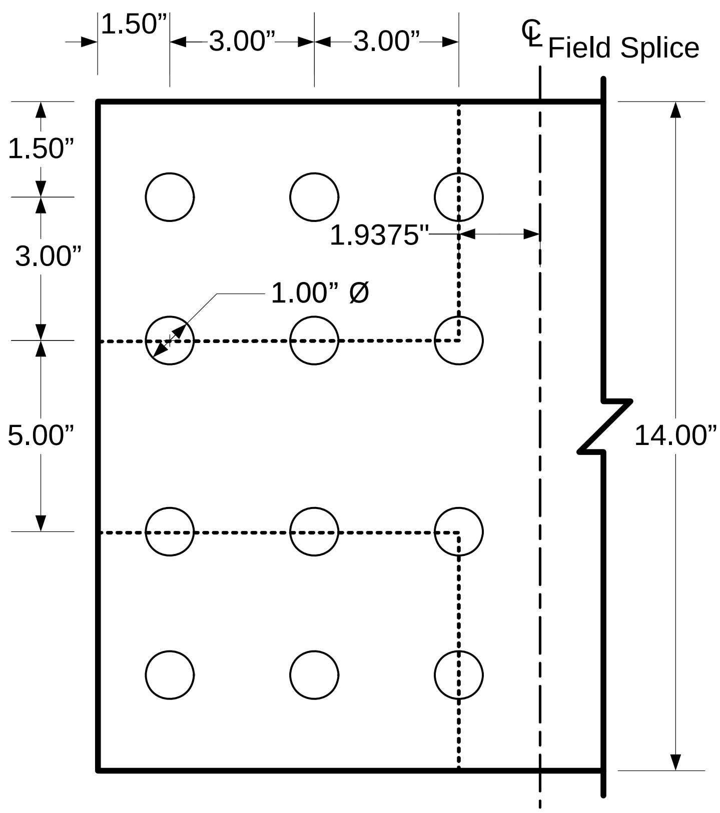


Figure E24-2.4-3
Outside Splice Plate - Failure Mode 2

The failure mode 2 calculations for the outside splice plates are are not shown since they are similar to those shown previously for failure mode 1. The final check for failure mode 2 is shown below.

Check:

$$R_r = 268.45 \text{ kips} > \boxed{\frac{P_{cu}}{2} = 179.63} \text{ kips} \quad \text{OK}$$

Inside splice plates:

The inside splice plates will now be checked for block shear. See Figure E24-2.4-4 for the assumed failure mode:

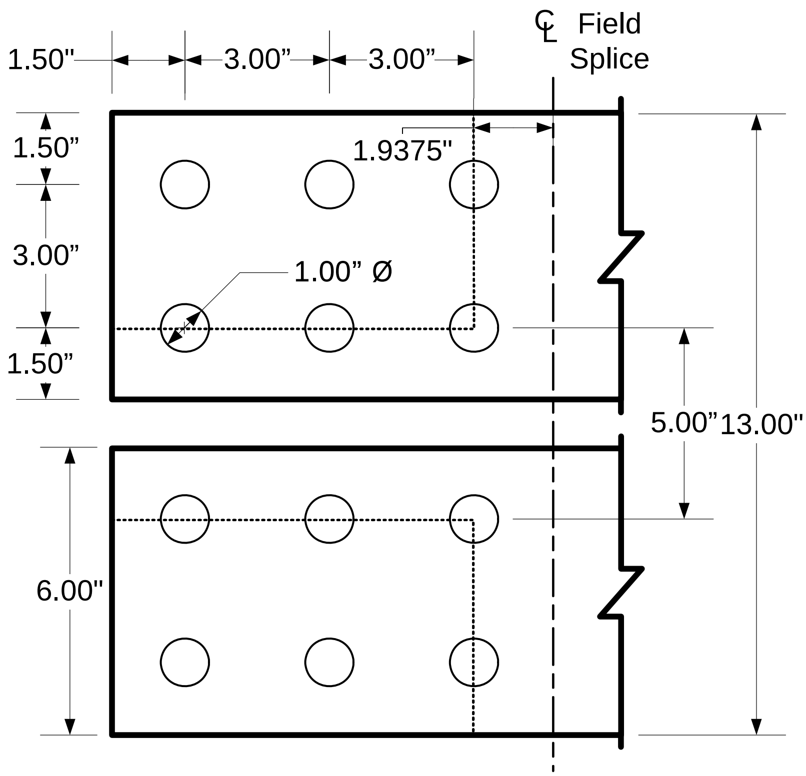


Figure E24-2.4-4
Inside Splice Plates - Block Shear Check

The calculations for the inside splice plates are not shown since they are similar to those shown previously for failure mode 1 and 2. The final check for the inside splice plates is shown below.

Check:

$$R_r = 306.80 \text{ kips} > \boxed{\frac{P_{cu}}{2} = 179.63} \text{ kips} \quad \text{OK}$$

Girder bottom flange:

The girder bottom flange will now be checked for block shear. See Figure E24-2.4-5 for the assumed failure mode:

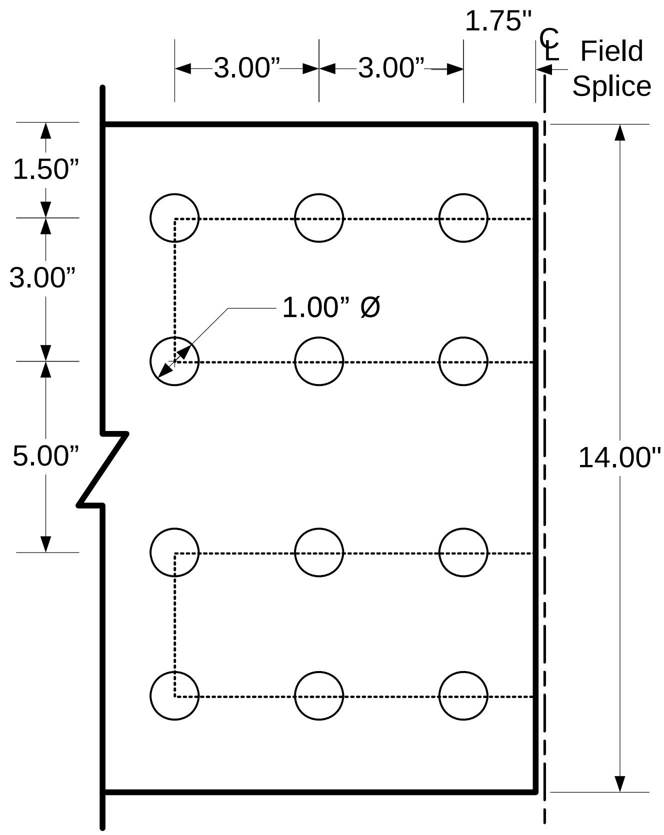


Figure E24-2.4-5
Bottom Flange - Block Shear Check

The calculations for the girder bottom flange are not shown since they are similar to those shown previously for the inside splice plates. The final check for the girder bottom flange is shown below.

Check:

$R_r = 736.19$ kips $>$ $P_{cu} = 359.25$ kips OK

It should be noted that although the block shear checks performed in this design example indicate an overdesign, the number of bolts cannot be reduced prior to checking shear on the bolts and bearing at the bolt holes. These checks are performed in what follows.

Net Section Fracture **LRFD [6.10.1.8]**

When checking flexural members at the strength limit state or for constructibility, the stress on the gross area of the tension flange due to the factored loads calculated without consideration of flange lateral bending, f_t, shall be satisfied at all cross-sections containing holes in the tension flange:

$$f_t \le 0.84 \cdot \left(\frac{A_n}{A_g}\right) \cdot F_u \le F_{yt}$$

Where:

A_n = Net area of the tension flange (in^2) **LRFD[6.8.3]**

A_g = Gross area of the tension flange (in^2)

F_{yt} = Specified minimum yield strength of a tension flange (ksi) **LRFD [C6.8.2.3]**

$A_n := (b_{flbL} - 4 \cdot d_{hole}) \cdot t_{flbL}$ $\boxed{A_n = 8.75}$ in^2

$A_g := t_{flbL} \cdot b_{flbL}$ $\boxed{A_g = 12.25}$ in^2

$F_u = 65.00$ ksi

$\boxed{0.84 \cdot \left(\frac{A_n}{A_g}\right) \cdot F_u = 39.00}$ ksi

$F_{yt} := 50$ ksi

$f_t := 37.5$ ksi (from Table E24-2.3-13)

$f_t \le 0.84 \cdot \left(\frac{A_n}{A_g}\right) \cdot F_u \le f_{yt}$ OK

Flange bolts - shear:

Determine the number of bolts for the bottom flange splice plates that are required to develop the Strength I design force in the flange in shear assuming the bolts in the connection have slipped and gone into bearing. A minimum of two rows of bolts should be provided to ensure proper alignment and stability of the girder during construction.

The Strength I flange design force used in this check was previously computed (reference Table E24-2.3-13):

$P_{cu} := 459.38$ kips

The factored resistance of an ASTM A325 7/8" diameter high-strength bolt in shear must be determined, assuming the threads are excluded from the shear planes. For this case, the number of bolts required to provide adequate shear strength is determined by assuming the design force acts on two shear planes, known as double shear.

The nominal shear resistance, R_n, is computed first as follows **LRFD [6.13.2.7]**:

$$R_n := (0.48 \cdot A_b \cdot F_{ub} \cdot N_s)$$

Where:

A_b = Area of the bolt corresponding to the nominal diameter (in^2)

F_{ub} = Specified minimum tensile strength of the bolt (ksi) **LRFD [6.4.3]**

N_s = Number of shear planes per bolt

$A_b := \frac{\pi}{4} \cdot d_{bolt}^{\ 2}$ $\boxed{A_b = 0.60}$ in^2

$F_{ub} := Fu_{bolt}$ $\boxed{F_{ub} = 120.00}$ ksi

$N_s := 2$

$R_n := 2 \cdot (0.48 \cdot A_b \cdot F_{ub})$ $\boxed{R_n = 69.27}$ kips

The factored shear resistance now follows:

$\phi_s = 0.80$ (see E24-2.1)

$R_u := \phi_s \cdot R_n$ $\boxed{R_u = 55.42}$ kips

When bolts carrying loads pass through fillers 0.25 inches or more in thickness in axially loaded connections, including girder flange splices, either **LRFD [6.13.6.1.5]**:

The fillers shall be extended beyond the gusset or splice material and shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler.

or

The fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the Strength Limit State, specified in **LRFD [6.13.2.2]**, is reduced by an appropriate factor:

In this design example, the reduction factor approach will be used. The reduction factor, R, per the specifications is:

$$R := \left(\frac{1+\gamma}{1+2\gamma}\right)$$

Where:

γ = A_f / A_p

A_f = Sum of the area of the fillers on the top and bottom of the connected plate (in^2)

A_p = Smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in²)

Sum of the area of the fillers on the top and bottom of the connected plate:

$A_f := b_{fill} t_{fill}$ $\boxed{A_f = 7.00}$ in²

The smaller of either the connected plate area (i.e., girder flange) or the sum of the splice plate areas on the top and bottom of the connected plate determines A_p.

Bottom flange area:

$b_{flbL} = 14.00$ in

$t_{flbL} = 0.88$ in

$A_{p1} := (b_{flbL}) \cdot (t_{flbL})$ $\boxed{A_{p1} = 12.25}$ in²

Sum of splice plate areas is equal to the gross areas of the inside and outside splice plates:

$A_{gross\_in} = 6.00$ in

$A_{gross\_out} = 6.13$ in

$A_{p2} := A_{gross\_in} + A_{gross\_out}$ $\boxed{A_{p2} = 12.13}$ in²

The minimum of the areas is:

$A_p := \min(A_{p1}, A_{p2})$ $\boxed{A_p = 12.13}$ in²

Therefore:

$\gamma := \frac{A_f}{A_p}$ $\boxed{\gamma = 0.58}$

The reduction factor is determined to be:

$R_{fill} := \left(\frac{1+\gamma}{1+2\gamma}\right)$ $\boxed{R_{fill} = 0.73}$

To determine the total number of bolts required for the bottom flange splice, divide the applied Strength I flange design force by the reduced allowable bolt shear strength:

$R := R_u \cdot R_{fill}$ $\boxed{R = 40.57}$ kips

The number of bolts required per side is:

$N := \frac{P_{cu}}{R}$ $\boxed{N = 11.32}$

The minimum number of bolts required on each side of the splice to resist the maximum Strength I flange design force in shear is twelve.

Flange bolts - slip resistance:

Bolted connections for flange splices shall be designed as slip-critical connections for the Service II flange design force, or the flange design force from constructibility, whichever governs **LRFD [6.13.6.1.4a]**. In this design example, the Service II flange force controls (see E24-2.3).

When checking for slip of the bolted connection for a flange splice with inner and outer splice plates, the slip resistance should always be determined by dividing the flange design force equally to the two slip planes regardless of the ratio of the splice plate areas **LRFD [C6.13.6.1.4c]**. Slip of the connection cannot occur unless slip occurs on both planes.

From Table E24-2.3-15, the Service II bottom flange design force is:

$P_s := 161.49$ kips

The factored resistance for slip-critical connections, R_n, is calculated from **LRFD [6.13.2.2 & 6.13.2.8]**:

$R_r := R_n$

$R_n := K_h \cdot K_s \cdot N_s \cdot P_t$

Where:

K_h = Hole size factor **LRFD [Table 6.13.2.8-2]**

K_s = Surface condition factor **LRFD [Table 6.13.2.8-3]**

N_s = Number of slip planes per bolt

P_t = Minimum required bolt tension (kips) **LRFD [Table 6.13.2.8-1]**

Determine the factored resistance per bolt assuming a Class B surface condition for the faying surface, standard holes (which are required per **LRFD [6.13.6.1.4a]**) and two slip planes per bolt:

Class B surfaces are unpainted blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings **LRFD [6.13.2.8]**.

$K_h := 1.0$

$K_s := 0.50$

$N_s := 2$

$P_t := 39.0$ kips

$R_r := K_h \cdot K_s \cdot N_s \cdot P_t$ $\boxed{R_r = 39.00}$ kips

The minimum number of bolts required to prevent slip is:

$$N := \frac{P_s}{R_r}$$ $\boxed{N = 4.14}$

Use:

$N := 5$ bolts < N = 12 bolts determined previously to satisfy the bolt shear requirements.

Therefore, the number of bolts required for the bottom-flange splice is controlled by the bolt shear requirements. Arrange the bolts in three rows of four bolts per line with no stagger.

Flange bolts - minimum spacing **LRFD [6.13.2.6.1]**:

The minimum spacing between centers of bolts in standard holes shall be no less than three times the diameter of the bolt.

$d_{bolt} = 0.875$ in

$s_{min} := 3 \cdot d_{bolt}$ $\boxed{s_{min} = 2.63}$ in

$s := 3.00$ in (see Figures E24-2.4-2 thru E24-2.4-5)

The minimum spacing requirement is satisfied.

Flange bolts - maximum spacing for sealing **LRFD [6.13.2.6.2]**:

For a single line adjacent to a free edge of an outside plate or shape (for example, the bolts along the edges of the plate parallel to the direction of the applied force):

$$s \le (4.0 + 4.0 \cdot t) \le 7.0$$

Where:

t = Thickness of the thinner outside plate or shape (in)

$t_{out} = 0.4375$ in

Maximum spacing for sealing:

$\boxed{4.0 + 4.0 \cdot t_{out} = 5.75}$ in

$s \le 5.75 \le 7.00$ OK

Next, check for sealing along the free edge at the end of the splice plate. The bolts are not staggered, therefore the applicable equation is:

$$s \le (4.00 + 4.00 \cdot t) \le 7.00$$

Maximum spacing along the free edge at the end of the splice plate (see Figures E24-2.4-2 thru E24-2.4-5):

$s_{end} := 5.00$ in

Maximum spacing for sealing:

$\boxed{4.0 + 4.0 \cdot t_{out} = 5.75}$ in

$s_{end} \le 5.75 \le 7.00$ OK

Flange bolts - maximum pitch for stitch bolts **LRFD [6.13.2.6.3]**:

The maximum pitch requirements are applicable only for mechanically fastened built-up members and will not be applied in this example.

Flange bolts - edge distance **LRFD [6.13.2.6.6]**:

The minimum required edge distance is measured as the distance from the center of any bolt in a standard hole to an edge of the plate. For a 7/8" diameter bolt measured to a sheared edge, the minimum edge distance is 1 1/2" **LRFD [Table 6.13.2.6.6-1]**. Referring to Figures E24-2.4-2 thru E24-2.4-5, it is clear that the minimum edge distance specified for this example is 1 1/2" and thus satisfies the minimum requirement.

The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or five inches.

$8 \cdot t \le 5.00$ in

$t := t_{out}$ $\boxed{t_{out} = 0.4375}$ in

$\boxed{8 \cdot t_{out} = 3.50}$ in

The maximum distance from the corner bolts to the corner of the splice plate or girder flange is equal to (reference Figure E24-2.4-5):

$\boxed{\sqrt{1.50^2 + 1.75^2} = 2.30}$ in

$2.30 \cdot in \le 3.50 \cdot in$ OK

Flange bolts - bearing at bolt holes **LRFD [6.13.2.9]**:

Check bearing of the bolts on the connected material under the maximum Strength I Limit State design force. The maximum Strength I bottom flange design force from Table E24-2.3-13 is:

$P_{cu} := 459.38$

The design bearing strength of the connected material is calculated as the sum of the bearing strengths of the individual bolt holes parallel to the line of the applied force.

The element of the bottom flange splice that controls the bearing check in this design example is the outer splice plate.

To determine the applicable equation for the calculation of the nominal resistance, the clear distance between holes and the clear end distance must be calculated and compared to the value of two times the nominal diameter of the bolt. This check yields:

$d_{bolt} = 0.875$ in $\boxed{2 \cdot d_{bolt} = 1.75}$ in

For the bolts adjacent to the end of the splice plate, the edge distance is 1 1/2". Therefore, the clear end distance between the edge of the hole and the end of the splice plate:

$d_{hole} = 1.00$

$L_{c_1} := 1.50 - \frac{d_{hole}}{2}$ $\boxed{L_{c_1} = 1.00}$ in

The center-to-center distance between bolts in the direction of the force is three inches. Therefore, the clear distance between edges of adjacent holes is computed as:

$L_{c_2} := 3.00 - d_{hole}$ $\boxed{L_{c_2} = 2.00}$ in

For standard holes, where either the clear distance between holes or the clear end distance is less than twice the bolt diameter, the nominal resistance, R_n, is taken as:

$R_n := 1.2 \cdot L_c \cdot t \cdot F_u$

Where:

L_c = Clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in)

For the outside splice plate:

$t_{out} = 0.4375$ in

$F_u = 65.00$ in

The nominal resistance for the end row of bolt holes is computed as follows:

$R_{n_1} := 4 \cdot \left(1.2 \cdot L_{c_1} \cdot t_{out} \cdot F_u\right)$ $\boxed{R_{n_1} = 136.50}$ kips

The nominal resistance for the remaining bolt holes is computed as follows:

$R_{n_2} := 8 \cdot \left(1.2 \cdot L_{c_2} \cdot t_{out} \cdot F_u\right)$ $\boxed{R_{n_2} = 546.00}$ kips

The total nominal resistance of the bolt holes is:

$R_n := R_{n_1} + R_{n_2}$ $\boxed{R_n = 682.50}$ kips

$\phi_{bb} = 0.80$

$R_r := \phi_{bb} \cdot R_n$ $\boxed{R_r = 546.00}$ kips

Check:

$\boxed{\frac{P_{cu}}{2} = 229.69}$ kips < $R_r = 546.00$ kips OK

Fatigue of splice plates **LRFD [6.6.1]**:

Check the fatigue stresses in the base metal of the bottom flange splice plates adjacent to the slip-critical connections. Fatigue normally does not govern the design of the splice plates, and therefore, an explicit check is not specified. However, a fatigue check of the splice plates is recommended whenever the combined area of the inside and outside flange splice plates is less than the area of the smaller flange at the splice.

From Table E24-2.3-16, the factored fatigue stress range at the midthickness of the bottom flange is:

$\Delta f_{fact} := 8.81$ ksi

For load-induced fatigue considerations, each detail shall satisfy:

$\gamma \cdot (\Delta f) \leq \Delta F_n$

Where:

γ = Load factor for fatigue I load combination **LRFD [Table 3.4.1-1]**

(Δf) = Force effect, live load stress range due to the passage of the fatigue load (ksi) **LRFD [3.6.1.4]**

ΔF_n = Nominal fatigue resistance (ksi) **LRFD [6.6.1.2.5]**

$\gamma := 1.50$

$\gamma(\Delta f) := \Delta f_{fact}$ $\boxed{\gamma(\Delta f) = 8.81}$ ksi

For the fatigue I load combination:

$\Delta F_n = \blacksquare \cdot \Delta F_{TH}$

Where:

ΔF_{TH} = Constant-amplitude fatigue threshold (ksi) **LRFD [Table 6.6.1.2.5-3]**

$\Delta F_{TH} := 16$ ksi

$\Delta F_n := \Delta F_{TH}$ $\boxed{\Delta F_n = 16.00}$ ksi (governs)

Check that the following is satisfied:

$\Delta f_{fact} \leq \Delta F_n$ $\Delta f_{fact} = 8.81$ ksi < $\Delta F_n = 16.00$ ksi OK

Control of permanent deflection - splice plates **LRFD [6.10.4.2]**:

A check of the flexural stresses in the splice plates at the Service II Limit State is not explicitly specified in the specifications. However, whenever the combined area of the inside and outside flange splice plates is less than the area of the smaller flange at the splice (which is the case for the bottom flange splice in this example), such a check is recommended.

The maximum Service II flange force in the bottom flange is taken from Table E24-2.3-15:

$P_S = 161.49$ kips

The following criteria will be used to make this check **LRFD [6.10.4.2]**. The equation presented is for both steel flanges of a composite section assuming no lateral flange bending:

$$f_f \le 0.95 \cdot R_h \cdot F_{yf}$$

Where:

f_f = Flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending (ksi)

R_h = Hybrid factor **LRFD [6.10.1.10.1]**

F_{yf} = Specified minimum yield strength of a flange (ksi) **LRFD [6.7.7.3]**

$F_{yf} = 50.00$ ksi

The flange force is equally distributed to the inner and outer splice plates due to the areas of the flanges being within 10 percent of each other:

$$P := \frac{P_S}{2} \qquad \boxed{P = 80.75} \text{ kips}$$

The resulting stress in the outside splice plate is:

$$f\_out := \frac{P}{A_{gross\_out}} \qquad \boxed{f\_out = 13.18} \text{ ksi}$$

$$f\_out = 13.18 \text{ ksi} < \boxed{0.95 \cdot F_{yf} = 47.50} \text{ ksi} \quad \text{OK}$$

The resulting stress in the inside splice plates is:

$$f\_in := 13.46 \text{ ksi} < \boxed{0.95 \cdot R_h \cdot F_{yf} = 47.50} \text{ ksi} \quad \text{OK}$$

E24-2.5 Design Top Flange Splice

The top flange splice is designed using the same procedures and methods presented in this design example for the bottom flange splice.

E24-2.6 Compute Web Splice Design Loads

Web splice plates and their connections shall be designed for shear, the moment due to the eccentricity of the shear at the point of splice, and the portion of the flexural moment assumed to be resisted by the web at the point of the splice **LRFD [6.13.6.1.4b]**.

Girder shear forces at the splice location:

A summary of the unfactored shears at the splice location from the initial trial of the girder design are listed below. The live loads include impact and distribution factors.

Dead load shears:

Noncomposite:

$V_{NDL} := -58.3$ kips

Composite:

$V_{CDL} := -7.7$ kips

Future wearing surface:

$V_{FWS} := -7.3$ kips

Live Load shears:

HL-93 positive:

$V_{PLL} := 14.7$ kips

HL-93 negative:

$V_{NLL} := -94.0$ kips

Fatigue positive:

$V_{PFLL} := 5.2$ kips

Fatigue negative:

$V_{NFLL} := -34.2$ kips

Web moments and horizontal force resultant **LRFD [C6.13.6.1.4b]**:

Because the portion of the flexural moment assumed to be resisted by the web is to be applied at the mid-depth of the web, a horizontal design force resultant, H_{uw}, must also be applied at the mid-depth of the web to maintain equilibrium. The web moment and horizontal force resultant are applied together to yield a combined stress distribution equivalent to the unsymmetrical stress distribution in the web. For sections with equal compressive and tensile stresses at the top and bottom of the web (i.e., with the neutral axis located at the mid-depth of the web), H_{uw} will equal zero.

In the computation of the portion of the flexural moment assumed to be resisted by the web, M_{uw}, and the horizontal design force resultant, H_{uw}, in the web, the flange stresses at the midthickness of the flanges are conservatively used. This allows use of the same stress values for both the flange and web splices, which simplifies the calculations. It is important to note that the flange stresses are taken as signed quantities in determining M_{uw} and H_{uw} (positive for tension; negative for compression).

The moment, M_{uv}, due to the eccentricity of the design shear, V_{uw}, is resisted solely by the web and always acts about the mid-depth of the web (i.e., horizontal force resultant is zero). This moment is computed as:

$$M_{uv} := V_{uw} \cdot e$$

Where:

e = The distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration (in)

$$e := 1.9375 + \frac{3.00}{2}$$ (see Figure E24-2.7-1) $\boxed{e = 3.4375}$ in

The total web moment for each load case is computed as follows:

$$M_{total} := M_{uw} + M_{uv}$$

In general, and in this example, the web splice is designed under the conservative assumption that the maximum moment and shear at the splice will occur under the same loading condition.

Strength I Limit State:

Design shear **LRFD [6.13.6.1.4b]**:

For the Strength I Limit State, the girder web factored shear resistance is required when determining the design shear **LRFD [6.10.9.2]**. Assume an unstiffened web at the splice location. The nominal shear resistance, V_n, is as follows:

$$V_n := C \cdot V_p$$

Where:

C = Ratio of the shear-buckling resistance to the shear yield strength in accordance with **LRFD [6.10.9.3.2]**, with the shear-buckling coefficient, k, taken equal to 5.0

V_p = Plastic shear force (kips)
= $0.58F_{yw}Dt_w$

F_{yw} = Specified minimum yield strength of a web (ksi)

D = Clear distance between flanges (in)

t_w = Web thickness (in)

$k := 5.0$

$E := 29000$ ksi

$F_{yw} := F_y$ $\boxed{F_{yw} = 50.00}$ ksi

$D = 54.00$ in (see Figure E24-2.1-1)

$t_w = 0.50$ in (see Figure E24-2.1-1)

Compare:

$$\boxed{\frac{D}{t_w} = 108.00}$$

to the values for:

$$\boxed{1.12 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 60.31}$$

and

$$\boxed{1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 75.39}$$

Based on the computed value of D/t_w, use the following equation to determine C **LRFD [6.10.9.3.2]**:

$$C := \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) \qquad \boxed{C = 0.39}$$

The nominal shear resistance, V_n, is computed as follows:

$V_p := 0.58 \cdot F_{yw} \cdot D \cdot t_w$ $\boxed{V_p = 783.00}$ kips

$V_n := C \cdot V_p$ $\boxed{V_n = 305.64}$ kips

The factored shear resistance, V_r, now follows:

$V_r := \phi_v \cdot V_n$ $\boxed{V_r = 305.64}$ kips

At the strength limit state, the design shear, V_{uw}, shall be taken as **LRFD [6.13.6.1.4b]**:

If:

$V_u < 0.5\ \phi_v V_n$, then:

then:

$$V_{uw} := 1.5 \cdot V_u$$

Otherwise:

$$V_{uw} := \frac{V_u + (\phi_v \cdot V_n)}{2}$$

The shear due to the Strength I loading at the point of splice, V_u, is computed from the girder shear forces at the splice location listed at the beginning of this section.

For the Strength I Limit State, the factored shear for the positive live load is:

$$V_{upos} := 0.90 \cdot (V_{NDL} + V_{CDL}) + 1.75 \cdot V_{PLL}$$ $\boxed{V_{upos} = -33.67}$ kips

For the Strength I Limit State, the factored shear for the negative live load is:

$$V_{uneg} := 1.25 \cdot (V_{NDL} + V_{CDL}) + 1.50 \cdot V_{FWS} + 1.75 \cdot V_{NLL}$$

$\boxed{V_{uneg} = -257.95}$ kips (controls)

Therefore:

$$V_u := |V_{uneg}|$$ $\boxed{V_u = 257.95}$ kips

Since V_u exceeds one-half of $\phi_v V_n$:

$$V_{uw} := \frac{V_u + (\phi_v \cdot V_n)}{2}$$ $\boxed{V_{uw} = 281.80}$ kips

Web moments and horizontal force resultants:

Case 1 - Dead load + positive live load:

For the loading condition with positive live load, the controlling flange was previously determined to be the bottom flange. The maximum elastic flexural stress due to the factored loads at the midthickness of the controlling flange, f_{cf}, and the design stress for the controlling flange, F_{cf}, were previously computed for this loading condition. From Table E24-2.3-13:

$f_{cf} := 18.32$ ksi

$F_{cf} := 37.50$ ksi

For the same loading condition, the concurrent flexural stress at the midthickness of the noncontrolling (top) flange, f_{ncu}, was previously computed. From Table E24-2.3-14:

$f_{ncf} := 0.33$ ksi

Therefore, the portion of the flexural moment, M_{uw}, assumed to be resisted by the web is computed as:

$$M_{uw} := \frac{t_w \cdot D^2}{12} \cdot |R_h \cdot F_{cf} - R_{cf} \cdot f_{ncf}|$$

Where:

F_{cf}	=	Design stress for the controlling flange at the point of splice specified in **LRFD [6.13.6.1.4c]**; positive for tension, negative for compression (ksi)
R_{cf}	=	The absolute value of the ratio of Fcf to the maximum flexural stress, fcf, due to the factored loads at the midthickness of the controlling flange at the point of splice, as defined in **LRFD [6.13.6.1.4c]**
f_{ncf}	=	Flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with f_{cf}; positive for tension, negative for compression (ksi)

$R_h = 1.00$

$$R_{cf} := \left|\frac{F_{cf}}{f_{cf}}\right| \qquad \boxed{R_{cf} = 2.05}$$

Compute the portion of the flexural moment to be resisted by the web:

$$M_{uw\_str\_pos} := \frac{t_w \cdot D^2}{12} \cdot \left|R_h \cdot F_{cf} - R_{cf} \cdot f_{ncf}\right| \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{uw\_str\_pos} = 372.85} \text{ kip-ft}$$

The total web moment is:

$$M_{tot\_str\_pos} := M_{uw\_str\_pos} + \left(V_{uw} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_str\_pos} = 453.57} \text{ kip-ft}$$

Compute the horizontal force resultant (the variables included in this equation are as defined for $M_{w\_str\_pos}$):

$$H_{uw\_str\_pos} := \frac{t_w \cdot D}{2} \cdot \left(R_h \cdot F_{cf} + R_{cf} \cdot f_{ncf}\right) \qquad \boxed{H_{uw\_str\_pos} = 515.37} \text{ kips}$$

The above value is a signed quantity, positive for tension and negative for compression.

Case 2 - Dead load + negative live load:

The calculations at the Strength I Limit State for case 2 are not shown since they are similar to those shown previously for case 1. The final web moment and horizontal force resultant for case 2 are shown below.

The total web moment is:

$$M_{tot\_str\_neg} := M_{uw\_str\_neg} + \left(V_{uw} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_str\_neg} = 759.38} \text{ kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_str\_neg} := \frac{t_w \cdot D}{2} \cdot \left(R_h \cdot F_{cf} + R_{cf} \cdot f_{ncf}\right) \qquad \boxed{H_{uw\_str\_neg} = -107.62} \text{ kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

Service II Limit State:

Design shear:

As a minimum, for checking slip of the web splice bolts, the design shear shall be taken as the shear at the point of splice under the Service II Limit State, or the shear from constructibility, whichever governs **LRFD [6.13.6.1.4b]**. In this design example, the Service II shear controls (see previous discussion in section E24-2.3).

For the Service II Limit State, the factored shear for the positive live load is (ignore future wearing surface):

$$V_{ser\_pos} := 1.00 \cdot V_{NDL} + 1.00 \cdot V_{CDL} + 1.30 \cdot V_{PLL} \qquad \boxed{V_{ser\_pos} = -46.89} \text{ kips}$$

For the Service II Limit State, the factored shear for the negative live load is (include future wearing surface):

$$V_{ser\_neg} := 1.00 \cdot V_{NDL} + 1.00 \cdot V_{CDL} + 1.00 \cdot V_{FWS} + 1.30 \cdot V_{NLL}$$

$$\boxed{V_{ser\_neg} = -195.50} \text{ kips (governs)}$$

Therefore:

$$V_{w\_ser} := \left|V_{ser\_neg}\right| \qquad \boxed{V_{w\_ser} = 195.50} \text{ kips}$$

Web moments and horizontal force resultants **LRFD [C6.13.6.1.4b]**:

$$M_{uw\_ser} := \frac{t_w \cdot D^2}{12} \cdot \left|f_s - f_{os}\right|$$

$$H_{uw\_ser} := \frac{t_w \cdot D}{2} \cdot \left(f_s + f_{os}\right)$$

Where:

f_s = Maximum Service II midthickness flange stress for the load case considered (i.e., positive or negative live load) (ksi)

f_{os} = Service II midthickness flange stress in the other flange, concurrent with f_s

Case 1 - Dead load + positive live load:

The maximum midthickness flange flexural stress for the load case with positive live load moment for the Service II Limit State occurs in the bottom flange. From Table 2E2.34-11:

$f_{s\_bot\_pos} := 13.18$ ksi

$f_{os\_top\_pos} := 0.71$ ksi

Therefore, for the load case of positive live load:

$$M_{uw\_ser\_pos} := \frac{t_w \cdot D^2}{12} \cdot \left| f_{s\_bot\_pos} - f_{os\_top\_pos} \right| \cdot \left(\frac{1}{12} \right)$$

$\boxed{M_{uw\_ser\_pos} = 126.26}$ kip-ft

$$M_{tot\_ser\_pos} := M_{uw\_ser\_pos} + \left(V_{w\_ser} \cdot e \right) \cdot \left(\frac{1}{12} \right)$$

$\boxed{M_{tot\_ser\_pos} = 182.26}$ kip-ft

Compute the horizontal force resultant:

$$H_{uw\_ser\_pos} := \frac{t_w \cdot D}{2} \cdot \left(f_{s\_bot\_pos} + f_{os\_top\_pos} \right)$$

$\boxed{H_{uw\_ser\_pos} = 187.52}$ kips

The above value is a signed quantity, positive for tension, and negative for compression.

Case 2 - Dead load + negative live load:

The maximum midthickness flange flexural stress for the load case with negative live load moment for the Service II Limit State occurs in the bottom flange. From Table E24-2.3-11:

$f_{s\_bot\_neg} := -12.33$ ksi

$f_{os\_top\_neg} := 1.94$ ksi

Therefore:

$$M_{uw\_ser\_neg} := \frac{t_w \cdot D^2}{12} \cdot \left| f_{s\_bot\_neg} - f_{os\_top\_neg} \right| \cdot \left(\frac{1}{12} \right)$$

$\boxed{M_{uw\_ser\_neg} = 144.48}$ kip-ft

The total web moment is:

$$M_{tot\_ser\_neg} := M_{uw\_ser\_neg} + \left(V_{w\_ser} \cdot e \right) \cdot \left(\frac{1}{12} \right)$$

$\boxed{M_{tot\_ser\_neg} = 200.49}$ kip-ft

Compute the horizontal force resultant:

$$H_{uw\_ser\_neg} := \frac{t_w \cdot D}{2} \cdot \left(f_{s\_bot\_neg} + f_{os\_top\_neg} \right)$$

$\boxed{H_{uw\_ser\_neg} = -140.27}$ kips

The above value is a signed quantity, positive for tension, and negative for compression.

Fatigue I Limit State:

Fatigue of the base metal adjacent to the slip-critical connections in the splice plates may be checked as specified in **LRFD [Table 6.6.1.2.3-1]** using the gross section of the splice plates and member **LRFD [C6.13.6.1.4a]**. However, the areas of the web splice plates will often equal or exceed the area of the web to which it is attached (the case in this design example). Therefore, fatigue will generally not govern the design of the splice plates, but is carried out in this example for completeness.

Design shear:

For the Fatigue I Limit State, the factored shear for the positive live load is:

$V_{fat\_pos} := 1.50 \cdot V_{PFLL}$ $\boxed{V_{fat\_pos} = 7.80}$ kips

For the Fatigue I Limit State, the factored shear for the negative live load is:

$V_{fat\_neg} := 1.50 \cdot V_{NFLL}$ $\boxed{V_{fat\_neg} = -51.30}$ kips

Web moments and horizontal force resultants:

The portion of the flexural moment to be resisted by the web and the horizontal force resultant are computed from equations similar to **LRFD [Equations C6.13.6.1.4b-1 & 6.13.6.1.4b-2]**, respectively, with appropriate substitutions of the stresses in the web caused by the fatigue-load moment for the flange stresses in the equations. Also, the absolute value signs are removed to keep track of the signs. This yields the following equations:

$$M_{uw} := \frac{t_w \cdot D^2}{12} \cdot \left(f_{botweb} - f_{topweb}\right)$$

$$H_{uw} := \frac{t_w \cdot D}{2} \cdot \left(f_{botweb} + f_{topweb}\right)$$

Case 1 - Positive live load:

The factored stresses due to the positive live load moment for the Fatigue I Limit State at the top and bottom of the web, from Table E24-2.3-12, are:

$f_{topweb\_pos} := -0.20$ ksi

$f_{botweb\_pos} := 4.96$ ksi

Therefore:

$$M_{uw\_fat\_pos} := \frac{t_w \cdot D^2}{12} \cdot \left(f_{botweb\_pos} - f_{topweb\_pos}\right) \cdot \left(\frac{1}{12}\right)$$

$\boxed{M_{uw\_fat\_pos} = 52.25}$ kip-ft

The total web moment is:

$$M_{tot\_fat\_pos} := M_{uw\_fat\_pos} + \left(V_{fat\_pos} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_fat\_pos} = 54.48} \text{ kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_fat\_pos} := \frac{t_w \cdot D}{2} \cdot \left(f_{botweb\_pos} + f_{topweb\_pos}\right) \qquad \boxed{H_{uw\_fat\_pos} = 64.26} \text{ kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

Case 2 - Negative live load:

The factored stresses due to the negative live load moment for the Fatigue I Limit State at the top and bottom of the web, from Table E24-2.3-12, are:

$f_{botweb\_neg} := -3.77$ ksi

$f_{topweb\_neg} := 0.15$ ksi

Therefore:

$$M_{uw\_fat\_neg} := \frac{t_w \cdot D^2}{12} \cdot \left(f_{botweb\_neg} - f_{topweb\_neg}\right) \cdot \left(\frac{1}{12}\right)$$

$$\boxed{M_{uw\_fat\_neg} = -39.69} \text{ kip-ft}$$

The total web moment is:

$$M_{tot\_fat\_neg} := M_{uw\_fat\_neg} + \left(V_{fat\_neg} \cdot e\right) \cdot \left(\frac{1}{12}\right) \qquad \boxed{M_{tot\_fat\_neg} = -54.39} \text{ kip-ft}$$

Compute the horizontal force resultant:

$$H_{uw\_fat\_neg} := \frac{t_w \cdot D}{2} \cdot \left(f_{botweb\_neg} + f_{topweb\_neg}\right) \qquad \boxed{H_{uw\_fat\_neg} = -48.87} \text{ kips}$$

The above value is a signed quantity, positive for tension, and negative for compression.

E24-2.7 Design Web Splice

Web splice configuration:

Two vertical rows of bolts with sixteen bolts per row will be investigated. The typical bolt spacings, both horizontally and vertically, are as shown in Figure E24-2.7-1. The outermost rows of bolts are located 4 1/2" from the flanges to provide clearance for assembly (see the *AISC Manual of Steel Construction* for required bolt assembly clearances). The web is spliced symmetrically by plates on each side with a thickness not less than one-half the thickness of the web. Assume 3/8" x 48" splice plates on each side of the web. No web fill plate is necessary for this example.

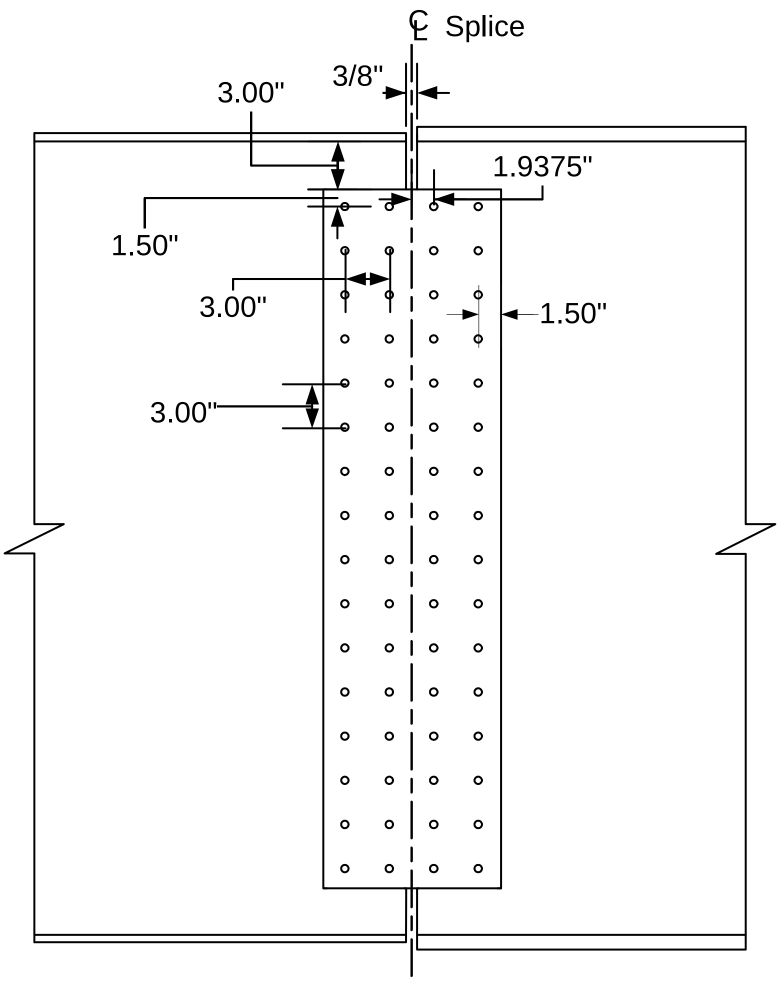


Figure E24-2.7-1
Web Splice

Web bolts - minimum spacing **LRFD [6.13.2.6.1]**:

This check is only dependent upon the bolt diameter, and is therefore satisfied for a three inch spacing per the check for the flange bolts from E24-2.4.

Web Bolts - Maximum Spacing for Sealing **LRFD [6.13.2.6.2]**:

The maximum spacing of the bolts is limited to prevent penetration of moisture in the joints.

For a single line adjacent to a free edge of an outside plate or shape (for example, the bolts along the edges of the plate parallel to the direction of the applied force):

$$s \le (4.0 + 4.0 \cdot t) \le 7.0$$

$t_{wp} := 0.375$ in

Maximum spacing for sealing:

$\boxed{4.0 + 4.0 \cdot t_{wp} = 5.50}$ in

$3.0 \le 5.5 \le 7.00$ OK

Web bolts - maximum pitch for stitch bolts **LRFD [6.13.2.6.3]**:

The maximum pitch requirements are applicable only for mechanically fastened built-up members and will not be applied in this example.

Web bolts - edge distance **LRFD [6.13.2.6.6]**:

Referring to Figure E24-2.7-1, it is clear that the minimum edge distance specified for this example is 1 1/2" and thus satisfies the minimum requirement **LRFD [Table 6.13.2.6.6-1]**.

The maximum edge distance shall not be more than eight times the thickness of the thinnest outside plate or five inches.

$t := t_{wp}$ $\boxed{t_{wp} = 0.3750}$ in

The maximum edge distance allowable is:

$8 \cdot t_{wp} = 3.00$

The maximum distance from the corner bolts to the corner of the splice plate or girder flange is equal to (reference Figure E24-2.7-1):

$\boxed{\sqrt{1.50^2 + 1.50^2} = 2.12}$ in

and satisfies the maximum edge distance requirement.

$2.12 \cdot in \le 2.50 \cdot in$ OK

Web bolts - shear:

Calculate the polar moment of inertia, I_p, of the bolt group on each side of the centerline with respect to the centroid of the connection **LRFD [C6.13.6.1.4.b]**. This is required for determination of the shear force in a given bolt due to the applied web moments.

$$I_p := \frac{n \cdot m}{12}\left[s^2 \cdot \left(n^2 - 1\right) + g^2 \cdot \left(m^2 - 1\right)\right]$$

Where:

m = number of vertical rows of bolts

n = number of bolts in one vertical row

s = vertical pitch (in)

g = horizontal pitch (in)

$m := 2$

$n := 16$

$s := 3.00$ in

$g := 3.00$ in

The polar moment of inertia is:

$$I_p := \frac{n \cdot m}{12}\left[s^2 \cdot \left(n^2 - 1\right) + g^2 \cdot \left(m^2 - 1\right)\right] \qquad \boxed{I_p = 6192.00} \quad in^2$$

The total number of web bolts on each side of the splice, assuming two vertical rows per side with sixteen bolts per row, is:

$N_b := 32$

Strength I Limit State:

Under the most critical combination of the minimum design shear, moment and horizontal force, it is assumed that the bolts in the web splice have slipped and gone into bearing. The shear strength of an ASTM A325 7/8" diameter high-strength bolt in double shear, assuming the threads are excluded from the shear planes, was computed in E24-2.4 for Flange Bolts - Shear:

$R_u = 55.42$ kips

Case 1 - Dead load + positive live load:

The following forces were computed in E24-2.6:

$V_{uw} = 281.80$ kips

$M_{tot\_str\_pos} = 453.57$ kip-ft

$H_{uw\_str\_pos} = 515.37$ kips

The vertical shear force in the bolts due to the applied shear force:

$$P_{v\_str} := \frac{V_{uw}}{N_b} \qquad \boxed{P_{v\_str} = 8.81} \quad kips$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_str\_pos} := \frac{H_{uw\_str\_pos}}{N_b} \qquad \boxed{P_{H\_str\_pos} = 16.11} \quad kips$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

$$P_{Mv} := \frac{M_{total} \cdot x}{I_p}$$

and

$$P_{Mh} := \frac{M_{total}\, y}{I_p}$$

For the vertical component:

$$x := \frac{g}{2} \qquad \boxed{x = 1.50} \quad \text{in}$$

For the horizontal component:

$$y := \frac{15 \cdot s}{2} \qquad \boxed{y = 22.50} \quad \text{in}$$

Calculating the components:

$$P_{Mv\_str\_pos} := \frac{M_{tot\_str\_pos} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_str\_pos} = 1.32} \quad \text{kips}$$

$$P_{Mh\_str\_pos} := \frac{M_{tot\_str\_pos}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_str\_pos} = 19.78} \quad \text{kips}$$

The resultant bolt force for the extreme bolt is:

$$P_{r\_str\_pos} := \sqrt{\left(P_{v\_str} + P_{Mv\_str\_pos}\right)^2 \ldots + \left(P_{H\_str\_pos} + P_{Mh\_str\_pos}\right)^2} \qquad \boxed{P_{r\_str\_pos} = 37.28} \quad \text{kips}$$

Case 2 - Dead load + negative live load:

The calculations at the Strength I Limit State for case 2 are not shown since they are similar to those shown previously for case 1. The final check for case 2 is shown below.

The following forces were computed in 24E2.6:

$V_{uw} = 281.80$ kips

$M_{tot\_str\_neg} = 759.38$ kip-ft

$H_{uw\_str\_neg} = -107.62$ kip-ft

The vertical shear force in the bolts due to the applied shear force:

$$P_{v\_str} := \frac{V_{uw}}{N_b} \qquad \boxed{P_{v\_str} = 8.81} \quad \text{kips}$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_str\_neg} := \frac{\left|H_{uw\_str\_neg}\right|}{N_b} \qquad \boxed{P_{H\_str\_neg} = 3.36} \quad \text{kips}$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

Calculating the components:

$$P_{Mv\_str\_neg} := \frac{M_{tot\_str\_neg} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_str\_neg} = 2.21} \quad \text{kips}$$

$$P_{Mh\_str\_neg} := \frac{M_{tot\_str\_neg}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_str\_neg} = 33.11} \quad \text{kips}$$

The resultant bolt force is:

$$P_{r\_str\_neg} := \sqrt{\left(P_{v\_str} + P_{Mv\_str\_neg}\right)^2 \cdots + \left(P_{H\_str\_neg} + P_{Mh\_str\_neg}\right)^2} \qquad \boxed{P_{r\_str\_neg} = 38.10} \quad \text{kips}$$

The governing resultant bolt force is:

$$P_{r\_str} := \max\left(P_{r\_str\_pos}, P_{r\_str\_neg}\right) \qquad \boxed{P_{r\_str} = 38.10} \quad \text{kips}$$

Check:

$P_{r\_str} = 38.10$ kips $<$ $R_u = 55.42$ kips OK

Service II Limit State:

The factored slip resistance, R_r, for a 7/8" diameter high-strength bolt in double shear for a Class B surface and standard holes was determined from E24-2.4 to be:

$R_r := 39.00$ kips

Case 1 - Dead load + positive live load:

The following forces were computed in E24-2.6:

$V_{w\_ser} = 195.50$ kips

$M_{tot\_ser\_pos} = 182.26$ kip-ft

$H_{uw\_ser\_pos} = 187.52$ kips

The vertical shear force in the bolts due to the applied shear force:

$$P_{s\_ser} := \frac{V_{w\_ser}}{N_b} \qquad \boxed{P_{s\_ser} = 6.11} \quad \text{kips}$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_ser\_pos} := \frac{H_{uw\_ser\_pos}}{N_b} \qquad \boxed{P_{H\_ser\_pos} = 5.86} \quad \text{kips}$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

For the vertical component:

$x = 1.50$ in

$$P_{Mv\_ser\_pos} := \frac{M_{tot\_ser\_pos} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_ser\_pos} = 0.53} \quad \text{kips}$$

For the horizontal component:

$y = 22.50$ in

$$P_{Mh\_ser\_pos} := \frac{M_{tot\_ser\_pos}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_ser\_pos} = 7.95} \quad \text{kips}$$

The resultant bolt force is:

$$P_{r\_ser\_pos} := \sqrt{\left(P_{s\_ser} + P_{Mv\_ser\_pos}\right)^2 + \left(P_{H\_ser\_pos} + P_{Mh\_ser\_pos}\right)^2} \qquad \boxed{P_{r\_ser\_pos} = 15.32} \quad \text{kips}$$

Case 2 - Dead load + negative live load:

The calculations at the Service II Limit State for case 2 are not shown since they are similar to those shown previously for case 1. The final check for case 2 is shown below.

The following forces were computed in 24E2.6:

$V_{w\_ser} = 195.50$ kips

$M_{tot\_ser\_neg} = 200.49$ kip-ft

$H_{uw\_ser\_neg} = -140.27$ kips

The vertical shear force in the bolts due to the applied shear force:

$$P_{s\_ser} := \frac{V_{w\_ser}}{N_b} \qquad \boxed{P_{s\_ser} = 6.11} \quad \text{kips}$$

The horizontal shear force in the bolts due to the horizontal force resultant:

$$P_{H\_ser\_neg} := \frac{\left|H_{uw\_ser\_neg}\right|}{N_b} \qquad \boxed{P_{H\_ser\_neg} = 4.38} \quad \text{kips}$$

Determine the horizontal and vertical components of the bolt shear force on the extreme bolt due to the total moment in the web:

For the vertical component:

$$P_{Mv\_ser\_neg} := \frac{M_{tot\_ser\_neg} \cdot (x)}{I_p} \cdot (12) \qquad \boxed{P_{Mv\_ser\_neg} = 0.58} \quad \text{kips}$$

For the horizontal component:

$$P_{Mh\_ser\_neg} := \frac{M_{tot\_ser\_neg}(y)}{I_p} \cdot (12) \qquad \boxed{P_{Mh\_ser\_neg} = 8.74} \quad \text{kips}$$

The resultant bolt force is:

$$P_{r\_ser\_neg} := \sqrt{\left(P_{s\_ser} + P_{Mv\_ser\_neg}\right)^2 \cdots + \left(P_{H\_ser\_neg} + P_{Mh\_ser\_neg}\right)^2} \qquad \boxed{P_{r\_ser\_neg} = 14.73} \quad \text{kips}$$

The governing resultant bolt force is:

$$P_{r\_ser} := \max\left(P_{r\_ser\_pos}, P_{r\_ser\_neg}\right) \qquad \boxed{P_{r\_ser} = 15.32} \quad \text{kips}$$

Check:

$P_{r\_ser} = 15.32$ kips < $R_r = 39.00$ kips OK

Thirty-two 7/8" diameter high-strength bolts in two vertical rows on each side of the splice provides sufficient resistance against bolt shear and slip.

Shear yielding of splice plates **LRFD [6.13.6.1.4b]**:

Check for shear yielding on the gross section of the web splice plates under the Strength I design shear force, V_{uw}:

$V_{uw} = 281.80$ kips

The factored resistance of the splice plates is taken as **LRFD [6.13.5.3]**:

$$R_r := \phi_v \cdot R_n$$

$$R_n := 0.58 \cdot A_g \cdot F_y$$

The gross area of the web splice is calculated as follows:

Number of splice plates:

$N_{wp} := 2$

Thickness of plate:

$t_{wp} := 0.375$ in

Depth of splice plate:

$d_{wp} := 48$ in

$A_{gross\_wp} := N_{wp} \cdot t_{wp} \cdot d_{wp}$ $\boxed{A_{gross\_wp} = 36.00}$ in^2

The factored shear resistance is then:

$\phi_v = 1.0$

$R_r := \phi_v \cdot (0.58) \cdot (A_{gross\_wp}) \cdot (F_y)$ $\boxed{R_r = 1044.00}$ kips

Check:

$V_{uw} = 281.80$ kips < $R_r = 1044.00$ kips OK

Fracture and block shear rupture of the web splice plates **LRFD [6.13.6.1.4b]**:

Strength I Limit State checks for fracture on the net section of web splice plates and block shear rupture normally do not govern for plates of typical proportion. These checks are provided in this example for completeness.

From E24-2.6, the factored design shear for the Strength I Limit State was determined to be:

$V_{uw} = 281.80$ kips

Fracture on the net section **LRFD [C6.13.4]**:

Investigation of critical sections and failure modes, other than block shear, is recommended, including the case of a net section extending across the full plate width, and, therefore, having no parallel planes. This may be a more severe requirement for a girder flange or splice plate than the block shear rupture mode.

For this case, the areas of the plate resisting tension are considered to be zero.

$A_{tn} := 0.0$ in^2

Therefore, the factored resistance is:

$R_r := \phi_{bs} \cdot (0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_y \cdot A_{tn})$

Number of web plates:

$N_{wp} = 2$

Depth of the web plate:

$d_{wp} = 48.00$ in

Number of bolts along one plane:

$N_{fn} := 16$

Thickness of the web plate:

$t_{wp} = 0.3750$ in

Specified minimum tensile strength of the connected material:

$F_u := 65$ ksi

Diameter of the bolt holes:

$d_{hole} = 1.00$ in

Net area resisting shear:

$A_{vn} := N_{wp} \cdot (d_{wp} - N_{fn} \cdot d_{hole}) \cdot t_{wp}$ $\boxed{A_{vn} = 24.00}$ in^2

A_{vn} of the splice plates to be used in calculating the fracture strength of the splice plates cannot exceed eighty-five percent of the gross area of the plates **LRFD [6.13.5.2]**:

$A_{85} := 0.85 \cdot A_{gross\_wp}$ $\boxed{A_{gross\_wp} = 36.00}$ in^2

$A_{85} = 30.60$ in^2 > $A_{vn} = 24.00$ in^2 OK

The factored resistance is then:

$R_r := \phi_{bs} \cdot (0.58 \cdot F_u \cdot A_{vn})$ $\boxed{R_r = 723.84}$ kips

$R_r = 723.84$ kips > $V_{uw} = 281.80$ kips OK

Block shear rupture resistance **LRFD [6.13.4]**:

Connection plates, splice plates and gusset plates shall be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection.

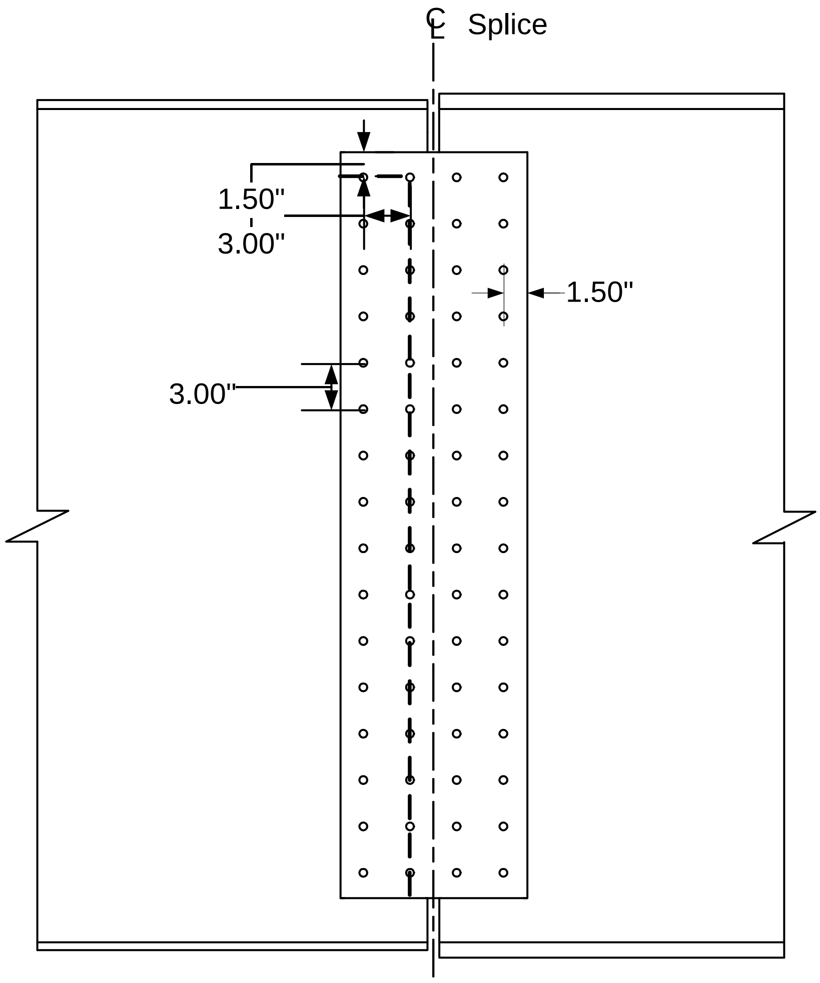


Figure E24-2.7-2
Block Shear Failure Mode - Web Splice Plate

Gross area along the plane resisting shear stress:

$$A_{vg} := N_{wp} \cdot (d_{wp} - 1.50) \cdot t_{wp} \qquad \boxed{A_{vg} = 34.88} \qquad in^2$$

Net area along the plane resisting shear stress:

$$A_{vn} := N_{wp} \cdot \left[d_{wp} - 1.50 - 15.50 \cdot (d_{hole}) \right] \cdot t_{wp} \qquad \boxed{A_{vn} = 23.25} \qquad in^2$$

Net area along the plane resisting tension stress:

$$A_{tn} := N_{wp} \cdot \left[1.50 + 3.0 - 1.5 \cdot (d_{hole}) \right] \cdot t_{wp} \qquad \boxed{A_{tn} = 2.25} \qquad in^2$$

$$U_{bs} := 1.0$$

$$R_{r1} := \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_u \cdot A_{vn} + U_{bs} \cdot F_u \cdot A_{tn}) = 818.22$$

$$R_{r2} := \phi_{bs} \cdot R_p \cdot (0.58 \cdot F_y \cdot A_{vg} + U_{bs} \cdot F_u \cdot A_{tn}) = 926.10$$

$R_r := \min(R_{r1}, R_{r2})$

$\boxed{R_r = 818.22}$ kips

Check:

$V_{uw} = 281.80$ kips < $R_r = 818.22$ kips OK

Flexural yielding of splice plates **LRFD [6.13.6.1.4b]**:

Check for flexural yielding on the gross section of the web splice plates for the Strength I Limit State due to the total web moment and the horizontal force resultant:

$$f := \frac{M_{Total}}{S_{pl}} + \frac{H_{uw}}{A_{gross\_wp}} \le \phi_f \cdot F_y$$

$\phi_f = 1.0$ (see E24-2.1)

Section modulus of the web splice plate:

$$S_{pl} := \frac{1}{6} \cdot A_{gross\_wp} \cdot d_{wp}$$

$\boxed{S_{pl} = 288.00}$ in^3

Case 1 - Dead load + positive live load:

$M_{tot\_str\_pos} = 453.57$ kip-ft

$H_{uw\_str\_pos} = 515.37$ kips

$$f_{str\_pos} := \frac{M_{tot\_str\_pos}}{S_{pl}} \cdot (12) + \frac{H_{uw\_str\_pos}}{A_{gross\_wp}}$$

$\boxed{f_{str\_pos} = 33.21}$ ksi

$f_{str\_pos} = 33.21$ ksi < $\boxed{\phi_f \cdot F_y = 50.00}$ ksi OK

Case 2 - Dead load + negative live load:

$M_{tot\_str\_neg} = 759.38$ kip-ft

$H_{uw\_str\_neg} = -107.62$ kips

$$f_{str\_neg} := \frac{M_{tot\_str\_neg}}{S_{pl}} \cdot (12) + \frac{\left|H_{uw\_str\_neg}\right|}{A_{gross\_wp}}$$

$\boxed{f_{str\_neg} = 34.63}$ ksi

$f_{str\_neg} = 34.63$ ksi < $\boxed{\phi_f \cdot F_y = 50.00}$ ksi OK

Web bolts - bearing resistance at bolt holes **LRFD [6.13.2.9]**:

Since the girder web thickness is less than twice the thickness of the web splice plates, the girder web will control for the bearing check.

Check the bearing of the bolts on the connected material for the Strength I Limit State assuming the bolts have slipped and gone into bearing. The design bearing strength of the girder web at the location of the extreme bolt in the splice is computed as the minimum resistance along the two orthogonal shear failure planes shown in Figure E24-2.7-3. The maximum force (vector resultant) acting on the extreme bolt is compared to this calculated strength, which is conservative since the components of this force parallel to the failure surfaces are smaller than the maximum force.

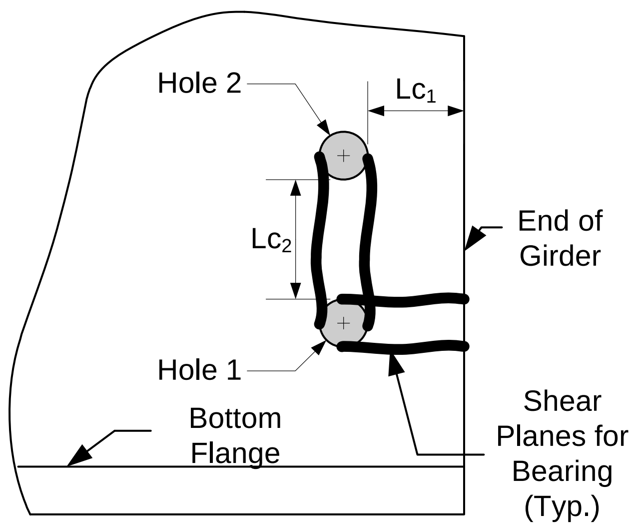


Figure E24-2.7-3
Bearing Resistance - Girder Web

To determine the applicable equation for the calculation of the nominal bearing resistance, the clear distance between holes and the clear end distance must be calculated and compared to the value of two times the nominal diameter of the bolt. This check yields:

$$\boxed{2 \cdot d_{bolt} = 1.75} \quad \text{in}$$

The edge distance from the center of the hole to the edge of the girder is taken as 1.75". Therefore, the clear distance between the edge of the hole and the edge of the girder is computed as follows **LRFD [6.13.2.6.6]**:

$$L_{c_1} := 1.75 - \frac{d_{hole}}{2} \qquad \boxed{L_{c_1} = 1.25} \quad \text{in}$$

The center-to-center distance between adjacent holes is 3". Therefore, the clear distance between holes is:

$$L_{c_2} := 3.00 - d_{hole} \qquad \boxed{L_{c_2} = 2.00} \quad \text{in}$$

For standard holes, where either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d **LRFD [6.13.2.9]**:

The nominal bearing resistance at the extreme bolt hole is as follows:

$R_n := 1.2 \cdot L_{c_1} \cdot t_w \cdot F_u$ $\boxed{R_n = 48.75}$ kips

The factored bearing resistance is:

$R_r := \phi_{bb} \cdot R_n$ $\boxed{R_r = 39.00}$ kips

The controlling minimum Strength I resultant bolt force was previously computed:

$P_{r\_str} = 38.10$ kips $<$ $R_r = 39.00$ kips OK

Fatigue of splice plates:

For load-induced fatigue considerations, each detail shall satisfy **LRFD [6.6.1.2.2]**:

$$\gamma \cdot (\Delta f) \leq \Delta F_n$$

Fatigue is checked at the bottom edge of the splice plates, which by inspection are subject to a net tensile stress.

The normal stresses at the bottom edge of the splice plates due to the total positive and negative fatigue-load web moments and the corresponding horizontal force resultants are as follows:

$$f := \frac{M_{total}}{S_{pl}} + \frac{H_w}{A_{gross\_wp}}$$

From previous calculations:

$S_{pl} = 288.00$ in^3

$A_{gross\_wp} = 36.00$ in^3

Case 1 - Positive live load:

$M_{tot\_fat\_pos} = 54.48$ kip-ft (see E24-2.6)

$H_{uw\_fat\_pos} = 64.26$ kips (see E24-2.6)

$f_{fat\_pos} := \frac{M_{tot\_fat\_pos}}{S_{pl}} \cdot (12) + \frac{H_{uw\_fat\_pos}}{A_{gross\_wp}}$ $\boxed{f_{fat\_pos} = 4.05}$ ksi

Case 2 - Negative live load:

$M_{tot\_fat\_neg} = -54.39$ kip-ft (see E24-2.6)

$H_{uw\_fat\_neg} = -48.87$ kips (see E24-2.6)

$f_{fat\_neg} := \frac{M_{tot\_fat\_neg}}{S_{pl}} \cdot (12) + \frac{H_{uw\_fat\_neg}}{A_{gross\_wp}}$ $\boxed{f_{fat\_neg} = -3.62}$ ksi

The total fatigue-load stress range at the bottom edge of the web splice plates is therefore:

$\gamma\Delta f := |f_{fat\_pos}| + |f_{fat\_neg}|$ $\boxed{\gamma\Delta f = 7.68}$ ksi

From E24-2.4, the fatigue resistance was determined as:

$\Delta F_n = 16.00$ ksi

$\gamma\Delta f = 7.68$ ksi $<$ $\Delta F_n = 16.00$ ksi OK

E24-2.8 Draw Schematic of Final Bolted Field Splice Design

Figure E24-2.8-1 shows the final bolted field splice as determined in this design example.

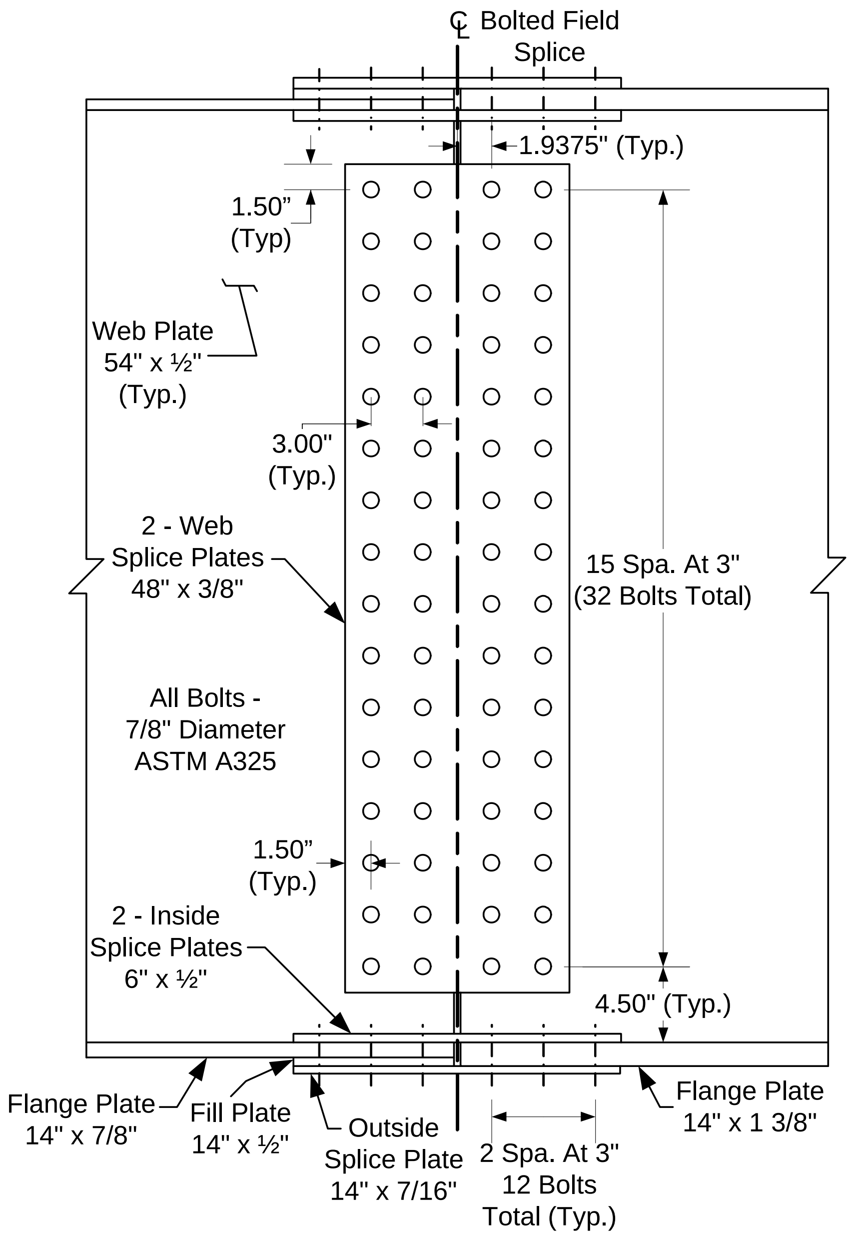


Figure E24-2.8-1
Final Bolted Field Splice Design

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30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *"Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,"* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *"Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,"* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *"Recommended Procedures for the Safety Performance Evaluation of Highway Features,"* represented a major update to the previously adopted report. The updates were based on significant changes in the vehicle fleet, the emergence of many new barrier designs, increased interest in matching safety performance to levels of roadway utilization, new policies requiring the use of safety belts, and advances in computer simulation and other evaluation methods.

NCHRP Report 350 differs from NCHRP Report 230 in the following ways: it is presented in all-metric documentation, it provides a wider range of test procedures to permit safety performance evaluations for a wider range of barriers, it uses a pickup truck as the standard test vehicle in place of a passenger car, it defines other supplemental test vehicles, it includes a broader range of tests to provide a uniform basis for establishing warrants for the application of roadside safety hardware that consider the levels of use of the roadway facility, it includes guidelines for selection of the critical impact point for crash tests on redirecting-type safety hardware, it provides information related to enhanced measurement techniques related to occupant risk, and it reflects a critical review of methods and technologies for safety-performance evaluation.

In May of 1997, a memorandum from Dwight A. Horne, the FHWA Chief of the Federal-Aid and Design Division, on the subject of “Crash Testing of Bridge Railings” was published. This memorandum identified 68 crash-tested bridge rails, consolidated earlier listings, and established tentative equivalency ratings that related previous NCHRP Report 230 testing to NCHRP Report 350 test levels.

In 2009, AASHTO published the *Manual for Assessing Safety Hardware* (MASH). MASH is an update to, and supersedes, NCHRP Report 350 for the purposes of evaluating new safety hardware devices. AASHTO and FHWA jointly adopted an implementation plan for MASH that stated that all highway safety hardware accepted prior to the adoption of MASH – using criteria contained in NCHRP Report 350 – may remain in place and may continue to be manufactured and installed. In addition, highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria. However, new highway safety hardware not previously evaluated must utilize MASH for testing and evaluation. MASH represents an update to crash testing requirements based primarily on changes in the vehicle fleet.

All bridge railings as detailed in the Wisconsin LRFD Bridge Standard Detail Drawings in Chapter 30, have been approved by FHWA per the crash tests as recommended in NCHRP Report 350. In order to use railings other than Bridge Office Standards, the railings must conform to MASH or must be crash tested rails which are available from the FHWA office. Any railings that are not crash tested must be reviewed by FHWA when they are used on a bridge, culvert, retaining wall, etc.

WisDOT policy states that railings that meet the criteria for Test Level 3 (TL-3) or greater shall be used on NHS roadways and all functional classes of Wisconsin structures (Interstate Highways, United States Highways, State Trunk Highways, County Trunk Highways, and Local Roadways) where the design speed exceeds 45 mph. Railings that meet Test Level 2 (TL-2) criteria may be used on non-NHS roadways where the design speed is 45 mph or less.

There may be unique situations that may require the use of a MASH or NCHRP Report 350 crash-tested railing of a different Test Level; a railing design using an older crash test methodology; or a modified railing system based on computer modeling, component testing, and or expert opinion. These unique situations will require an exception to be granted by the Bureau of Project Development and/or the Bureau of Structures. It is recommended that coordination of these unique situations occur early in the design process.

30.2 Railing Application

The primary purpose of bridge railings shall be to contain and redirect vehicles and/or pedestrians using the structure. In general, there are three types of bridge railings – Traffic Railings, Combination Railings, and Pedestrian Railings. The following guidelines indicate the typical application of each railing type:

1. Traffic Railings shall be used when a bridge is used exclusively for highway traffic.

   Traffic Railings can be composed of, but are not limited to: single slope concrete parapets, sloped face concrete parapets, vertical face concrete parapets, tubular steel railings, and timber railings.

2. Combination Railings can be used concurrently with a raised sidewalk on roadways with a design speed of 45 mph or less.

   Combination Railings can be composed of, but are not limited to: single slope concrete parapets with chain link fence, vertical face concrete parapets with tubular steel railings such as type 3T, and aesthetic concrete parapets with combination type C1-C6 railings.

3. Pedestrian Railings can be used at the outside edge of a bridge sidewalk when a Traffic Railing is used concurrently to separate highway and pedestrian traffic.

   Pedestrian Railings can be composed of, but are not limited to: chain link fence, tubular screening, vertical face concrete parapets with combination type C1-C6 or type 3T railings, and single slope concrete parapets.

See Figure 30.2-1 below for schematics of the three typical railing types.

Note that the railing types shown in Figure 30.2-1 shall be employed as minimums. At locations where a Traffic Railing is used at the traffic side of a sidewalk at grade, a Combination Railing may be used at the edge of deck in lieu of a Pedestrian Railing. At locations where a Combination Railing is used at the exterior edge of a raised sidewalk, a Traffic Railing may be used as an alternative as long as the requirements for Pedestrian Railings are met.

A “SS” or solid parapet shall be used on all grade separation structures and railroad crossings to minimize snow removal falling on the traffic below.

2. The sloped face parapet "LF" and “HF” parapets shall be used as Traffic Railings for rehabilitation projects only to match the existing parapet type. The sloped face parapets were crash-tested per NCHRP Report 230 and meet NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. The “51F” parapet shall only be used as a Traffic Railing on the median side of a structure when it provides a continuation of an approach 51 inch high median barrier.

4. The vertical face parapet “A” can be used for all design speeds. The vertical face parapet is recommended for use as a Combination Railing on raised sidewalks or as a Traffic Railing where the design speed is 45 mph or less. If the structure has a raised sidewalk on one side only, a sloped parapet should be used on the side opposite of the sidewalk. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the vertical face parapet can be used as a Pedestrian Railing. Under some circumstances, the vertical face parapet “A” can be used as a Traffic Railing for design speeds exceeding 45 mph with the approval of the Bureau of Structures Development Section. The vertical face parapet “A” was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. Aesthetic railings may be used if crash tested according to Section 30.1 or follow the guidance provided in Section 30.4. See Chapter 4 – Aesthetics for CSS considerations.

   The Texas style aesthetic parapet, type “TX”, can be used as a Traffic/Pedestrian Railing on raised sidewalks on structures with a design speed of 45 mph or less. For design speeds exceeding 45 mph, at locations where the parapet is protected by a Traffic Railing between the roadway and a sidewalk at grade, the type “TX” parapet can be used. This parapet is very expensive; however, form liners simulating the openings can be used to reduce the cost of this parapet. The type “TX” parapet was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. The type “PF” tubular railing, as shown in the Standard Details of Chapter 40, shall <u>not</u> be used on new bridge plans with a PS&E after 2013. This railing was not allowed on the National Highway System (NHS). The type “PF” railing was used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less.

7. Combination Railings, type “C1” through “C6”, are shown in the Standard Details and are approved as aesthetic railings attached to concrete parapets. The aesthetic additions are placed at least 5” from the crash-tested rail face per the Standard Details and have previously been determined to not present a snagging potential. Combination railing, type “3T”, without the recessed details on the parapet faces may be used when aesthetic details are not desired or when CSS funding is not available (see Chapter 4 – Aesthetics). These railings can only be used when the design

speed is 45 mph or less, or the railing is protected by a Traffic Railing between the roadway and a sidewalk at grade. The crash test criteria of the combination railings are based on the concrete parapets to which they are attached (i.e., if a type “C1” combination railing is attached to the top of a vertical face parapet type “A”, the parapet and railing combination meet crash test criteria for TL-4).

8. Chain Link Fence and Tubular Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between the roadway and a sidewalk at grade. Chain Link Fence, when attached to the top of a concrete Traffic Railing, can be used for design speeds exceeding 45 mph. Due to snagging and breakaway potential of the vertical spindles, Tubular Screening should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets (“A” or “SS”) as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: “Type H (insert railing type) railing shall not be used”. The combination railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. The type “W” railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 based on a May 1997 FHWA memorandum.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated

for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing was crash-tested per NCHRP Report 350 and meets criteria for TL-4.

13. Type “NY3/NY4” steel railings, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “NY3/NY4” railings may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “NY3/NY4” railings also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. The type “NY4” railing may be used on a raised sidewalk where the design speed is 45 mph or less. However, the type “NY” railings are not allowed for use on prestressed box girder bridges. These railings shall be considered where the Region requests an open railing. The type “NY” railings were crash-tested per NCHRP Report 350 and meet criteria for TL-4.

14. The type "F" steel railing, as shown in the Standard Details of Chapter 40, shall <u>not</u> be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less. Details in Chapter 40 are for <u>informational purposes only</u>.

15. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the *Facilities Development Manual (FDM) Standard Detail Drawings (SDD) 14b20*. Railing is not required on box culverts if the culvert is extended to provide an adequate clear zone as defined in *FDM 11-15-1*. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. The railing shall be provided only when it is cost effective as defined in *FDM Procedure 11-45-1*.

16. When the structure approach thrie beam is extended across the box culvert; refer to Standard Detail, Box Culvert Details for additional information. The minimum dimension between end of box and face of guard rail provides an acceptable rail deflection to prevent a vehicle wheel from traversing over the end of the box culvert. In almost every case, the timber posts with offset blocks and standard beam guard are used. Type "W" railing may be used for maintenance and box culvert extensions to mitigate the effect of structure modifications.

See the *FDM* for additional railing application requirements. See *11-45-1* and *11-45-2* for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See *11-35-1*, *Table 1.2* for requirements when barrier wall separation between roadway and sidewalk is necessary.

30.3 General Design Details

1. Epoxy coated bars are required for all concrete parapets, curbs, medians, and sidewalks.

2. Adhesive anchored parapets are allowed at interior Traffic Railing locations only when the adjacent exterior parapet is a crash test approved Traffic Railing per Section 30.2 (i.e., cast-in-place anchors are used at exterior parapet location). See Standard Details 30.10 and 30.14 for more information.

3. Sign structures, sign trusses, and monotubes shall be placed on top of railings to meet the working width and zone of intrusion dimensions noted in *FDM 11-45 Section 2.3.6.2.2* and *Section 2.3.6.2.3* respectively.

4. It is desirable to avoid attaching noise walls to bridge railings. However, in the event that noise walls are required to be located on bridge railings, compliance with the setback requirements stated in Section 30.4 and what is required in *FDM 11-45 Sections 2.3.6.2.2* and *2.3.6.2.3* is not required. *Note: WisDOT is currently investigating the future use of noise walls on bridge structures in Wisconsin.*

5. Temporary bridge barriers shall be designed in accordance with *FDM SDD 14b7*. Where temporary bridge barriers are being used for staged construction, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

6. Provide for expansion movement in tubular railings where expansion devices or concrete parapet deflection joints exist on the structure plan details. The tubular railing splice should be located over the joint and spaced evenly between railing posts. The tubular railing splice should be made continuous with a movable internal sleeve. If tubular railing is employed on conventional structures where expansion joints are likely to occur at the abutments only, the posts may be placed at equal spacing provided that no post is nearer than 2 feet from deflection joints in the parapet at the piers.

7. Refer to Standard Detail 30.07 – Vertical Face Parapet “A” – for detailing concrete parapet or sidewalk deflection joints. These joints are used based on previous experience with transverse deck cracking beneath the parapet joints.

8. Horizontal cracking has occurred in the past near the top of some concrete parapets which were slip formed. Similar cracking has not occurred on parapets cast in forms. Therefore, slip forming of bridge parapets shall not be allowed.

9. For beam guard type “W” railing, locate the expansion splice at a post or on either side of the expansion joint.

10. Sidewalks - If there is a Traffic Railing between the roadway and an at grade sidewalk, and the roadway side of the Traffic Railing is more than 11’-0” from the exterior edge of deck, access must be provided to the at grade sidewalk for the snooper truck to inspect the underside of the bridge. The sidewalk width must be

10’-0” clear between barriers, including fence (i.e., use a straight fence without a bend). For protective screening, the total height of parapet and fence need not exceed 8’-0”. The boom extension on most snooper trucks does not exceed 11’-0” so provision must be made to get the truck closer to the edge.

11. Where Traffic Railing is utilized between the roadway and an at grade sidewalk, early coordination with the roadway designer should occur to provide adequate clearances off of the structure to allow for proper safety hardware placement and sidewalk width. Additional clearance may be required in order to provide a crash cushion or other device to protect vehicles from the blunt end of the interior Traffic Railing off of the structure.

12. On shared-use bridges, fencing height and geometry shall be coordinated with the Region and the DNR (or other agencies) as applicable. Consideration shall be given to bridge use (i.e., multi-use/snowmobile may require vertical and horizontal clearances to allow grooming machine passage) and location (i.e., stream crossing vs. grade separation).

13. Per **LRFD [13.7.1.1]**, the use of raised sidewalks on structures shall be restricted to roadways with a design speed of 45 mph or less. The height of curbs for sidewalks is usually 6 inches. This height is more desirable than higher heights with regards to safety because it is less likely to vault vehicles. However, a raised curb is not considered part of the safety barrier system. On structure rehabilitations, the height of sidewalk may increase up to 8 inches to match the existing sidewalk height at the bridge approaches. Contact the Bureau of Structures Development Section if sidewalk heights in excess of 8 inches are desired. See Standard Detail 17.01 – Median and Raised Sidewalk Details – for typical raised sidewalk detail information.

14. Pedestrian loads, as described in the AASHTO LRFD Bridge Design Specifications, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

30.4 Railing Aesthetics

Railing aesthetics have become a key component to the design and delivery of bridge projects in Wisconsin. WisDOT Regions, local communities and their leaders use rail aesthetics to draw pedestrians to use the walkways on structures. With the increased desire to use, and frequency of use of aesthetics on railings, it has become increasingly important to set policy for railing aesthetics on bridge structures.

Railing aesthetics policies have been around for multiple decades. In the 1989 version of the AASHTO Standard Specifications, generalities were listed for use with designing bridge rails. Statements such as “Use smooth continuous barrier faces on the traffic side” and “Rail ends, posts, and sharp changes in the geometry of the railing shall be avoided to protect traffic from direction collision with the bridge rail ends” were used as policy and engineering judgment was required by each individual designer. This edition of the Standard Specifications aligned with NCHRP Report 350.

Caltrans conducted full-scale crash testing of various textured barriers in 2002. This testing was the first of its kind and produced acceptable railing aesthetics guidelines for single slope barriers for NCHRP Report 350 TL-3 conditions. Some of the allowable aesthetics were: sandblast textures with a maximum relief of 3/8”, geometric patterns inset into the face of the barrier 1” or less and featuring 45° or flatter chamfered or beveled edges, and any pattern or texture with a maximum relief of 2½” located 24” above the base of the barrier. Later in 2002, Harry W. Taylor, the Acting Director of the Office of Safety Design of FHWA, provided a letter to Caltrans stating that their recommendations were acceptable for use on all structure types.

In 2003, WisDOT published a paper titled, “Acceptable Community Sensitive Design Bridge Rails for Low Speed Streets & Highways in Wisconsin”. The goal of this paper was to streamline what railing aesthetics were acceptable for use on structures in Wisconsin. WisDOT policy at that time allowed vertical faced bridge rails in low speed applications to contain aesthetic modifications. For NHS structures, WisDOT allowed various types of texturing and relief based on crash testing and analysis. Ultimately, WisDOT followed many of the same requirements that were deemed acceptable by FHWA based on the Caltrans study in 2002.

NCHRP Report 554 – Aesthetic Concrete Barrier Design – was published in 2006 to (1) assemble a collection of examples of longitudinal traffic barriers exhibiting aesthetic characteristics, (2) develop design guidelines for aesthetic concrete roadway barriers, and (3) develop specific designs for see-through bridge rails. This publication serves as the latest design guide for aesthetic bridge barrier design and all bridge railings on structures in Wisconsin shall comply with the guidance therein.

<u>The application of aesthetics on bridge railings on structures in Wisconsin with a design speed exceeding 45 mph shall comply with the following guidance</u>:

1. All Traffic Railings shall meet the crash testing guidelines outlined in Section 30.1.

2. The top surface of concrete parapets shall be continuous without raised features (pilasters, pedestals, etc.) that potentially serve as snag points for vehicles or blunt

30.8 Railing Rehabilitation

The FHWA, in its implementation plan for MASH, requires that bridge railings on the NHS shall meet the requirements of MASH or NCHRP Report 350. In addition, FHWA states that “Agencies are encouraged to upgrade existing highway safety hardware that has not been accepted under MASH or NCHRP Report 350 during reconstruction projects, during 3R (Resurfacing, Restoration, Rehabilitation), or when the railing system is damaged beyond repair”.

WisDOT requirements for the treatment of existing railings for various project classifications are outlined in Table 30.8-1:

Project Classification	**Railing Rehabilitation**
Preventative Maintenance✳ (Resurfacing, Restoration) For letting dates after December 31, 2019: The compliance document will be MASH 2016 Edition	Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required. Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings where it is not feasible to install an approved railing. Coordination with BOS and BPD is required. NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350. Non-NHS Structures: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 be upgraded to comply with MASH or NCHRP Report 350.

<table>
<tr>
<td rowspan="2">3R✳✳
(Resurfacing, Restoration, Rehabilitation)

<u>For letting dates after December 31, 2019</u>:
The compliance document will be MASH 2016 Edition</td>
<td>If rehabilitation work, as part of the 3R project, is scheduled or performed which does not widen the structure nor affect the existing railing.</td>
<td>Replacement of bridge railing not in compliance with MASH or NCHRP Report 350 is recommended but not required provided the minimum rail height requirement is met. (Minimum rail height shall be 27” for roadway design speed of 45 mph or less and 32” for roadway design speed exceeding 45 mph.)

Existing railings – both in compliance and not in compliance – with MASH, NCHRP Report 350, and NCHRP Report 230 may be altered to improve the performance of the existing railings (i.e., raised to meet the minimum rail height requirement) where it is not feasible to install an approved railing. Coordination with BOS and BPD is required.

<u>NHS Structures</u>: Existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement shall be upgraded to comply with MASH or NCHRP Report 350.

<u>Non-NHS Structures</u>: It is strongly encouraged that existing railing that does not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and does not meet the minimum rail height requirement be upgraded to comply with MASH or NCHRP Report 350.</td>
</tr>
<tr>
<td>If rehabilitation work, as part of the 3R project, is scheduled or performed which widens the structure to either side, redecks (full-depth) any complete span of the structure, or if any work affecting the rail is done to the existing structure.</td>
<td>All railing on the structure must comply with MASH or NCHRP Report 350.

Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.</td>
</tr>
<tr>
<td colspan="2">4R (Resurfacing, Restoration, Rehabilitation, Reconstruction)

<u>For letting dates after December 31, 2019</u>:
The compliance document will be MASH 2016 Edition</td>
<td>All railing on the structure must comply with MASH or NCHRP Report 350.

Limited project by project exceptions may be granted based on coordination and input by the Bureau of Structures and the Wisconsin Division of FHWA Structures Engineer.</td>
</tr>
</table>

<u>Table 30.8-1</u>
WisDOT Requirements for Retrofitting/Upgrading Bridge Railings to Current Standards

✳ Examples of Preventative Maintenance projects include, but are not limited to:

1. Bridge deck work: Concrete deck repair, patching, and concrete overlays; asphaltic overlays; epoxy and polymer overlays; expansion joint replacement when done in

conjunction with an overlay or expansion joint elimination; chloride extraction; installation of a cathodic protection system.

2. Superstructure and substructure work: Steel structure cleaning and repainting, including complete repainting, zone painting, and spot painting with overcoat; structural repairs (except vehicle impact damage); bearing repair or replacement.

✳✳ Examples of 3R projects include, but are not limited to:

1. Bridge deck work: Bridge deck widenings and re-decks; expansion joint replacement when done in conjunction with an overlay or expansion joint elimination; approach slab replacement.

2. Superstructure and substructure work: Wing wall replacement; emergency bridge repair; structural repairs to railings based on vehicle impact damage;

The minimum railing height shall be measured from the top inside face of the railing to the top of the roadway surface at the toe of railing.

For all railing rehabilitations that require upgrades to comply with MASH or NCHRP Report 350, railings shall be employed as discussed in Section 30.2.

The following is a list of typical railing types that are in service on structures in Wisconsin. The <u>underlined</u> railings comply with MASH, NCHRP Report 350, or NCHRP Report 230 and may remain in service within rehabilitation projects. The *italicized* railings do not comply with MASH, NCHRP Report 350, or NCHRP Report 230 and shall be removed from service within rehabilitation projects.

1. <u>Single slope parapet “32SS”, “36SS”, “42SS”, “56SS”</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 specifications and meet crash test criteria for TL-4.

2. <u>Sloped face parapet "LF"</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

3. <u>Sloped face parapet "HF"</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

4. <u>Vertical face parapet “A”</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

5. <u>Aesthetic parapet “TX”</u>. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

6. Type “PF” tubular railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum. Standard Details in Chapter 40.

7. Type “H” railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-4 based on a May 1997 FHWA memorandum.

8. Timber Railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-2 based on a May 1997 FHWA memorandum.

9. Type "W" railing. Railing may be used for rehabilitation projects on non-NHS structures only. Crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 based on a May 1997 FHWA memorandum.

10. Type “M” railing. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 350 and meets TL-4

11. *Type “F” railing.* Railing may not be used for rehabilitation projects. Standard Details in Chapter 40 are for informational purposes only.

12. Sloped face parapet “B”. Railing may be used for rehabilitation projects. Crash-tested per NCHRP Report 230. Standard Details in Chapter 40.

The region shall contact the Bureau of Structures Development Section to determine the sufficiency of existing railings not listed above.

Rehabilitation or improvement projects to historically significant bridges require special attention. Typically, if the original railing is present on a historic bridge, it will likely not meet current crash testing requirements. In some cases, the original railing will not meet current minimum height and opening requirements. There are generally two different options for upgrading railings on historically significant bridges – install a crash-tested Traffic Railing to the interior side of the existing railing and leave the existing railing in place or replace the existing railing with a crash-tested Traffic Railing. Other alternatives may be available but consultation with the Bureau of Structures Development Section is required.

30.9 Railing Guidance for Railroad Structures

Per an April 2013 memorandum written by M. Myint Lwin, Director of the FHWA Office of Bridge Technology, bridge parapets, railings, and fencing shall conform to the following requirements when used in the design and construction of grade separated highway structures over railroads:

1. For NHS bridges over railroad:

   Bridge railings shall comply with AASHTO standards. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   However, railings for use on NHS bridges over railroads shall be governed by the railroad's standards, regardless of whether the bridge is owned by the railroad or WisDOT. For the case where an NHS bridge crosses over railroads operated by multiple authorities with conflicting parapet, railing, or fencing requirements, standards as agreed by the various railroad authorities and as approved by WisDOT shall be used.

2. For non-NHS bridges over railroad:

   Bridge railings shall comply with the policies outlined within this chapter. For Federal-aid highway projects, the designer shall follow normal bridge railing specifications, design standards, and guidelines.

   All federally funded non-NHS bridges including those over railroads shall be governed by WisDOT's policies outlined above, even if they differ from the railroad's standards.

30.10 References

1. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications.*

2. American Association of State Highway and Transportation Officials. *Manual for Assessing Safety Hardware*.

3. National Cooperative Highway Research Program. *NCHRP Report 554 – Aesthetic Concrete Barrier Design*.

4. State of California, Department of Transportation. *Crash Testing of Various Textured Barriers*.

5. National Cooperative Highway Research Program. *NCHRP Report 350 – Recommended Procedures for the Safety Performance Evaluation of Highway Features*.

6. State of Wisconsin, Department of Transportation. *Facilities Development Manual*.

7. State of Wisconsin, Department of Transportation. *Wisconsin Bicycle Facility Design Handbook.*

8. State of Wisconsin, Department of Transportation. *Memorandum of Understanding between Wisconsin Department of Transportation, Wisconsin County Highway Association, and Transportation Builders Association.*

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37.1 Structure Selection

Most pedestrian bridges are located in urban areas and carry pedestrian and/or bicycle traffic over divided highways, expressways and freeway systems. The structure type selected is made on the basis of aesthetics and economic considerations. A wide variety of structure types are available and each type is defined by the superstructure used. Some of the more common types are as follows:

- Concrete Slab
- Prestressed Concrete Girder
- Steel Girder
- Prefabricated Truss

Several pedestrian bridges are a combination of two structure types such as a concrete slab approach span and steel girder center spans. One of the more unique pedestrian structures in Wisconsin is a cable stayed bridge. This structure was built in 1970 over USH 41 in Menomonee Falls. It is the first known cable stayed bridge constructed in the United States. Generally, pedestrian bridges provide the designer the opportunity to employ long spans and medium depth sections to achieve a graceful structure.

Pedestrian boardwalks will not be considered “bridges” when their clear spans are less than or equal to 20 feet, and their height above ground and/or water is less than 10 feet. Boardwalks falling under these constraints will not be required to follow the design requirements in the WisDOT Bridge Manual, but will need to follow the standards established in the *Wisconsin Bicycle Facility Design Handbook.*

37.2 Specifications and Standards

The designer shall refer to the following related specifications:

- "*AASHTO LRFD Bridge Design Specifications*"
- *"AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges",* hereafter referred to as the *"Pedestrian Bridge Guide"*
- See *Standardized Special Provision (STSP)* titled "*Prefabricated Steel Truss Pedestrian Bridge LRFD*" for the requirements for this bridge type

For additional design information, refer to the appropriate Wisconsin Bridge Manual chapters relative to the structure type selected.

The pedestrian live load (PL) shall be as follows: (from "*Pedestrian Bridge Guide")*

- 90 psf **[Article 3.1]**
- Dynamic load allowance is not applied to pedestrian live loads **[Article 3.1]**

The vehicle live load shall be applied as follows: (from *"Pedestrian Bridge Guide")*

- Design for an occasional single maintenance vehicle live load (LL) **[Article 3.2]**

Clear Bridge Width (w)	Maintenance Vehicle
7 ft $\leq$ w $\leq$ 10 ft	H5 Truck (10,000 lbs)
w > 10 ft	H10 Truck (20,000 lbs)

- Clear bridge widths of less than 7 feet need not be designed for maintenance vehicles. **[Article 3.2]**
- The maintenance vehicle live load shall not be placed in combination with the pedestrian live load. **[Article 3.2]**
- Dynamic load allowance is not applied to the maintenance vehicle. **[Article 3.2]**
- Strength I Limit State shall be used for the maintenance vehicle loading. **[Article 3.2, 3.7]**

On Federal Aid Structures FHWA requests a limiting gradient of 8.33 percent (1:12) on ramps for pedestrian facilities to accommodate the physically handicapped and elderly as recommended by the "*American Standard Specifications for Making Buildings and Other Facilities Accessible to, and Usable by, the Physically Handicapped*". This is slightly flatter than the gradient guidelines set by AASHTO which states gradients on ramps should not be more than 15 percent and preferably not steeper than 10 percent.

The minimum inside clear width of a pedestrian bridge on a pedestrian accessible route is 8 feet. (*AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities, 2004).*

The width required is based on the type, volume, and direction of pedestrian and/or bicycle traffic.

The vertical clearance on the pedestrian bridge shall be a minimum of 10 feet for bicyclists' comfort and to allow access for maintenance and emergency vehicles. The Wisconsin Department of Natural Resources recommends a vertical clearance on the bridge of at least 12 feet to accommodate maintenance and snow grooming equipment on state trails. Before beginning the design of the structure, the Department of Natural Resources and the Bureau of Structures should be contacted for the vertical clearance requirements for all vehicles that require access to the bridge.

In addition, ramps should have rest areas or landings 5 feet to 6 feet in length which are level and safe. Rest area landings are mandatory when the ramp gradient exceeds 5 percent. Recommendations are that landings be spaced at 60 foot maximum intervals, as well as wherever a ramp turns. This value is based on a maximum gradient of 8.33 percent on pedestrian ramps, and placing a landing at every 5 feet change in vertical elevation. Also, ramps are required to have handrails on both sides. See Standard Details for handrail location and details.

Minimum vertical clearance for a pedestrian overpass can be found in the *Facilities Development Manual (FDM)* Procedure 11-35-1, Attachment 1.8 and 1.9. Horizontal clearance is provided in accordance with the requirement found in *(FDM)* Procedure 11-35-1, Attachment 1.5 and 1.6.

Live load deflection limits shall be in accordance with the provisions of **LRFD [2.5.2.6.2]** for the appropriate structure type.

Pedestrian loads, as described in the AASHTO LRFD Bridge Design Specifications, shall be used to not only design the pedestrian railings on the structure, but shall also be used to design stairway railings that are adjacent to the structure and are part of the contract.

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39.1 General

Signing is an integral part of the highway plan and as such is developed with the roadway and bridge design. Aesthetic as well as functional considerations are essential to sign structure design. Supporting sign structures should exhibit clean, light, simple lines which do not distract the motorist or obstruct view of the highway. In special situations sign panels may be supported on existing or proposed grade separation structures in lieu of an overhead sign structure. Aesthetically this is not objectionable if the sign does not extend below the girders or above the top of the parapet railing. Some of the more common sign support structures are shown in the following figure.

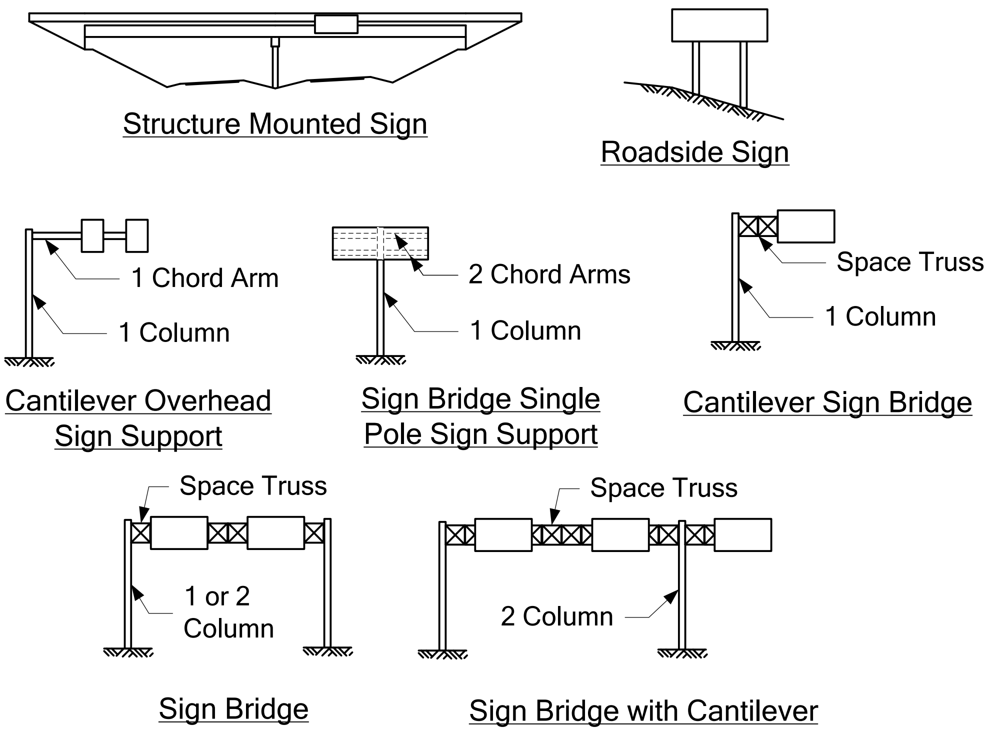

Figure 39.1-1
Sign Support Structures

39.1.1 Signs on Roadway

Roadside sign supports are located behind existing or planned guardrail as far as practical from the roadway out of the likely path of an errant vehicle. If roadside signs are located within the 30 foot corridor and not protected, break-away sign supports are detailed. Wisconsin has experienced that the upper hinge on ground mounted signs with break-away supports does not work and it is not used. Since FHWA has not approved this removal, the hinge is used on all federal projects. DMS, which includes both dynamic message signs and variable message signs, roadside sign type supports are to be protected by concrete barrier or guardrail. All overhead sign-column type supports are located at the edge of shoulder adjacent to the traveled roadway or placed behind barrier type guardrail. See the Facilities

Development Manual (FDM) 11-55-20.5 for information on shielding requirements. When protection is impractical or not desirable, the towers shall be designed with applicable extreme event collision loads in accordance to Section 13.4.10 of this manual.

Overhead sign structures, for new and replacement structures only, are to have a minimum vertical clearance of 18'-3" above the roadway. See FDM, Procedure 11-35-1 Attachment 1.9, for clearances relating to existing sign structures. The minimum vertical clearance is set slightly above the clearance line of the overpass structure for signs attached to existing structures.

39.1.2 Signs Mounted on Structures

Signs are typically installed along the major axes of a structure. Wisconsin has allowed sign attachment up to a maximum of a 20 degree skew. Any structure with greater skew requires mounting brackets to attach signs perpendicular to the roadway.

39.1.2.1 Signs Mounted on the Side of Structures

In addition to aesthetic reasons, signs attached to the side of bridge superstructures and retaining walls are difficult to inspect and maintain due to lack of access. Attachment accessories are susceptible to deterioration from de-icing chemicals, debris collection and moisture; therefore, the following guidance should be considered when detailing structure side mounted signs and related connections:

1. Limit the sign depth to a dimension equal to the bridge superstructure depth (including parapet) minus 3 inches.

2. Provide at least two point connections per supporting bracket.

3. Utilize cast-in-place anchor assemblies to attach sign supports onto new bridges and retaining walls.

4. Galvanized or stainless steel adhesive concrete masonry anchor may be used to attach new signs to the vertical face of an existing bridge or retaining wall for shear load application only. Overhead installation is not allowed. Reference Section 40.16 for applicable concrete masonry anchor requirements.

39.1.2.2 Overhead Structure Mounted Signs

Span deflections of the superstructure due to vehicle traffic are felt in overhead sign structures mounted on those bridges. The amount and duration of sign structure deflections is dependent on the stiffness of the girder and deck superstructure, the location of the sign on the bridge, and the ability of the sign structure to dampen those vibrations out; among others. These vibrations are not easily accounted for in design and are quite variable in nature. For these reasons, the practice of locating overhead sign structures onto bridges should be avoided whenever possible.

The following general guidance is given for those instances where locating a sign structure onto a bridge structure is unavoidable, which may be due to the length of the bridge, or a safety need to guide the traveling public to upcoming ramp exits or into specific lanes on the bridge.

1. Locate the sign structure support bases at pier locations.

2. Build the sign structure base off the top of the pier cap.

3. Provide set back of the tower support of the sign structure behind the back face of the parapet to preclude snagging of any vehicle making contact with the parapet.

4. Use single pole sign supports (equal balanced butterfly's) in lieu of cantilevered (with an arm on only one side of the vertical support) sign supports.

5. Consider the use of a Stockbridge type damper in the horizontal truss of these structures.

6. Do not straddle the pier leaving one support on the pier and one support off the pier in the case of skewed substructure units for full span sign bridges.

39.2 Specifications and Standards

Reference specifications for sign structures are as follows:

- AASHTO "*Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, 6th Edition*", and Interim Revisions
- AASHTO "*Standard Specifications for Highway Bridges, 17th Edition*"
- State of Wisconsin "*Standard Specifications for Highway and Structure Construction*"
- ASTM "*Standards of the American Society for Testing and Materials*"
- AWS D1.1 Structural Welding Code (Steel)
- AWS D1.2 Structural Welding Code (Aluminum)

Standard details for full span 4-chord galvanized steel sign bridge, design data and details for galvanized cantilever steel sign truss and footing are given on the Chapter 39 Standard Details.

Standard details for overhead sign support bases are provided in the Standard Detail Drawing (SDD) sheets of the FDM.

Standard design data and details for break-away sign supports and sign attachment are given on the A Series of the Sign Plate Manual.

39.3 Materials

Wisconsin has historically specified API Spec. 5L, grade 42 pipe as the primary material for the design of sign bridge chords and columns. However, due to supply shortage, API Spec. 5L, grade 46 and 52, ASTM A500 grades B and C, and ASTM A53 grade B types E and S round HSS or pipe (tubular shapes) are allowed as alternate materials for sign bridge truss main members (chords and columns) less than 10 inches diameter. API Spec. 5L, grade 42 remains the preferred material for single column on both full span and cantilever sign bridges due to the toughness requirement to address weldability, fatigue concerns and the non-redundant nature of these structures. All plates, bars and structural angles shall be ASTM A709 grade 36. ASTM A595 grade A, A572, and A1011 have been used by manufacturers to design round, tapered steel members for overhead sign support arms and uprights. When tubular shapes are used for overhead sign supports, they shall conform to the sign bridge requirements. Unless noted otherwise in the contract plans, all bolted connections for sign structures shall be made with direct tension indicating (DTI) washers and meet the applicable requirements of high strength A325 bolts as stated in Section 24.2 of this manual. More details can be found in the Standard Drawings and Standard Specifications Section 641.

WisDOT policy item:

Installation of flat washers in between faying surfaces of mast arm connection plates are not allowed.

39.4 Design Considerations

39.4.1 Signs on Roadway

Supports for roadside signs are of three types, depending upon the size and type of the sign to be supported. For small signs, the column supports are treated timber embedded in the ground. For larger type I signs and DMS, the columns shall be galvanized steel supported on cylindrical concrete footings. Currently, all steel column supports for roadside type I signs are designed to break-away upon impact, while DMS supports are protected and designed without a break-away system.

WisDOT policy item:

Type I break-away sign supports and foundations are design in accordance to the "AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals 1985". Standard design and support estimates are given in the A3 Series of the "Sign Plate Manual"

Wisconsin does not have standard design or details available for DMS roadside sign supports. Each support structure to be design by structure engineer, and the design must be in compliance with the applicable specifications listed in Section 39.2. An allowable soil pressure of 3.0 ksf shall be used to design the footings, unless subsurface condition is in question then investigation per Section 39.6.3 would be implemented to gather necessary design information. DMS sign supports and footings to be detailed with the Structure Plan Section of the contract.

39.4.2 Overhead Sign Structures

Sign structures for support of overhead signs consist of "sign bridges" and "overhead sign supports". Sign bridges are to be either a single column cantilever or butterfly, or a space truss sign bridge supported by one or two columns at each end. For cantilever sign bridge structures, the footing is a single cylindrical shaft with wings to prevent the overturning and twisting of the structure. For space trusses having one or two steel columns on an end, the footing is composed of two cylindrical caissons connected by a concrete cross-girder. The top surface of concrete foundations for all sign bridges is to be located 3' above the highest ground line at the foundation. Occasionally, some sign bridge columns are mounted directly on top of modified bridge parapets, pier caps and concrete towers instead of footings.

Sign bridges also include sign support members mounted directly onto structures. Sign attachments, such as galvanized steel I-beams and/or brackets, typically are anchored to the side of the bridge superstructure. A cantilever truss attached to the side of retaining walls (without a vertical column) is also common.

Similar to sign bridges, all overhead sign supports have single galvanized steel column supported on a cylindrical caisson footing or on top of bridge elements. Cross members can be one chord (monotube), two chord without web elements, or planar truss in either cantilever or full span structure.

The following design data is employed for designing steel sign bridges and overhead sign supports.

Wind Velocity = 90 mph based on the 3-second gust wind speed map and its corresponding methods to find wind pressure.

Dead Load = Wt. of Sign, supporting structure, catwalk, railings and lights.

Ice Load = 3 psf to one face of sign and around surface of members.

Group Load	Load Combination	% of Allowable Stress
I	DL	100
II	DL + W	133
III	DL + Ice + (1/2)W[a]	133
IV	Fatigue	[c]

Table 39.4-1
Group Load Combinations

[a] Minimum Wind Load = 25 psf

[c] See Fatigue section of AASHTO for fatigue loads and stress range limits.

Wind Components	Normal	Transverse
Combination 1	1.0	0.2
Combination 2	0.6	0.3

Table 39.4-2
Wind Components

WisDOT policy item:

Fatigue group loads application is exempt on four chord full span sign bridges supporting type I and II signs with truss type towers mounted on concrete footings and full span overhead sign supports mounted on top of standard concrete bases.

Steel cantilevered sign bridge structures (four chord structures carrying type I and II signage) detailed on Standard 39.10 thru 39.12 are classified, for purposes of fatigue design, as Category 1 structures. These cantilevered support structures are designed to resist Natural Wind Gust and Truck-Induced Gust wind effects, but not designed for Galloping wind effects due to the substantial stiffness and satisfactory performance history in this state. The design of these structures are in accordance to the AASHTO "Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 4th Edition" and interim Revisions.

All other sign structures shall be designed with applicable design specifications as stated in Section 39.2 of this manual.

Steel cantilever sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. Columns are made from pipe sections. The minimum thickness for the members is indicated on the steel cantilever Standard detail.

Steel full span sign bridge trusses are designed and fabricated from tubular shapes for chords and angle shapes for web members. The minimum thickness of steel web members is 3/16 inch and 0.216 inches for chord members. The connections of web members to chords are designed for bolting or shop welding to allow the contractor the option to either galvanize individual members or complete truss sections after fabrication. The columns are either steel pipe or tubular shape sections with web members (planar truss), see Section 39.3 for additional details. Steel base plates are used for anchor rod support attachment.

When butt welding round sections, a back-up plate is required since the plates can only be welded from one side. The plate must be of adequate width for film to be used during weld inspection. The exposed weld is ground smooth for appearance as well as fatigue. Shop splices typically done with the use of butt weld, but quality on large weld is difficult to achieve and not economical. Therefore, designers are advised to limit weld size to 5/8", and avoid shop splice on single column member whenever possible.

Aluminum sign bridges are currently not being designed for new structures. Rehabilitation and repair type work may require use of aluminum members and shall be allowed in these limited instances. The following guidelines apply to aluminum structures in the event of repair and rehabilitation type work.

Aluminum sign bridge trusses are designed and fabricated from tubular shapes shop welded together in sections. The minimum thickness of truss chords is ¼ inch and the minimum outside diameter is 4 inches. The recommended minimum ratios of "d/D" between the outside diameters "d" of the web members and "D" of chord members is 0.4. A cast aluminum base plate is required to connect the aluminum columns to the anchor rods. AASHTO Specifications require damping or energy absorbing devices on aluminum overhead sign support structures to prevent vibrations from causing fatigue failures. Damping devices are required before and after the sign panels are erected on all aluminum sign bridges. Stock-bridge type dampers are recommended.

Install permanent signs to sign structures at the time of erection. If the signs are not available, install sign blanks to control vibration. For sign bridges, blanks are attached to a minimum of one-fourth the truss length near its center. The minimum depth of the blanks is equal to the truss depth plus 24 inches. The blanks are to be installed to project an equal dimension beyond the top and bottom chord members. Overhead sign support blanks are equal to the same sizes and at the same locations as the permanent signs. Contact BOS Structures Design Section at 608-267-2869 for further guidance on other vibration controlling methods.

Do not add catwalks to new sign bridges unless they contain DMS over traffic. Catwalks add additional cost to a structure and present a maintenance issue. They can be added if a decision is made to light the signs in the future. Design structures with type I and II signs for a 20’-3” (2’-0” additional) vertical clearance when they are located in a continuous median freeway lighting area, for new and replacement sign bridges only. Structures with DMS may require larger vertical clearance to the bottom of the sign depending on the type of catwalk being designed for future installation. The sign bridge should be structurally designed to support a catwalk for those cases when the additional clearance is provided for possible future attachment. Additional accommodations for potential future lighting include providing hand holes in the columns, rodent screens and conduits in the concrete bases.

For structures that are not located in continuous median freeway lighting areas or do not contain DMS, the additional structure height should not be utilized. Therefore, the design vertical clearance should be 18’-3” for new and replacement sign bridges only. No hand holes, rodent screens or conduits shall be installed on the structure in this case. However, all DMS sign bridges require hand holes, rodent screens and conduits.

Brackets, if required, for maintenance of light units are required to support a 2'-3” wide catwalk grating and a collapsible aluminum handrail. Brackets and handrailing for type I and II signs are fabricated from aluminum sections, whereas DMS support brackets are made of galvanized steel. Catwalk grating and toe plates are fabricated from steel and shall be galvanized.

Contract plans shall include details and notes indicating if hand holes are required on one or both towers of the sign bridge.

Overhead sign supports are typically not lit, nor do they require sign maintenance. Therefore, do not detail a catwalk on this type of structure. Also do not detail hand holes, rodent screens and conduits unless the structure is designed to carry an LED changeable message sign, traffic signals or luminaires.

Design of all Sign Bridge structures should reflect some provision for the possibility of adding signs in the future (additional sign area). Consideration should include the number of lanes, possible widening of roadway into the median or shoulder areas, and use of diagrammatic signs to name a few. The truss design should reflect sizing the chords for maximum force at the center of the span. The design of the tower columns and truss webs should allow for signs being placed (say sometime in the future) more skewed to one side than the other. Columns should be selected the same size (outside diameter x thickness) for each side and the design shall reflect different lengths on either side as required by site conditions.

The design sign area and maximum sign depth dimensions for type I and II signs shall be explicitly listed with the design data in the contract plans. Use 3 psf dead load for these types of signs. Provide manufacturer overall DMS dimensions in the plans along with the total weight of the signs. Other loads such as Catwalks, lights and associated attachments must also be included in the overall design data in the contract plans.

The following guidance is recommended for estimating design sign areas.

1. Type I and II signs on full span sign bridges, design sign area equals the largest value resulting from the four requirements below:

    a. Total actual sign area.

b. Two (2) times the controlling tower tributary sign area. Tributary area is computed based on the application of the lever rule on a simply supported truss.

c. Twelve (12) times the number of lanes times the maximum sign depth. The number of lanes is defined as the clear roadway width (including median and shoulders) divided by 12 and rounding down to the nearest whole number.

d. Maximum sign depth times 60% of the span length (center to center of tower).

For design purposes, the standard sign depth shall be limited between 12’-0” and 16’-0”. Therefore, vertical clearance and column lengths are to be sized with sign depth not less than 12’-0”, unless requested otherwise in the structure survey report. Mega projects with series of sign bridges may deviate from the above requirements provided that coordination is made with the BOS Structures Design Section.

2. Type I and II maximum design sign area for galvanized steel cantilever sign truss is detailed in Standard 39.10. Sizing the column length and vertical clearance with 12’-0” sign depth is recommended for future accommodation.

3. DMS sign bridges should be designed with the actual sign dimensions in addition to those of type I and II signs and catwalk as applied.

4. Overhead sign supports are generally designed with the actual sign dimensions and locations. Exception to the approach may be granted to structures with anticipated change in signage.

39.5 Structure Selection Guidelines

Sign structures are composed of “sign bridges” and “overhead sign supports”. Either type of sign structure can be configured to be a cantilever sign structure (one column to a horizontal truss arm) or a full-span sign structure (two towers, one on each end of the span). Single pole (butterfly) is another type of sign bridge (chords centered on a single column). Roadside sign supports are an exception to the above naming convention.

“Sign Bridges” generally carry type I and II signs, and occasionally DMS. These are large sign structures with sign depths ranging from 5’-0” or less to 18’-0” in the case of large diagrammatic signs. Butterfly sign bridges are limited to 218 sq. ft. of sign area per side. Total sign areas accommodated are up to 264 sq. ft. on cantilever sign bridges. Total sign areas accommodated on full span sign bridges range from 250 to over 1000 sq. ft. These ranges are for approximate guide only. Butterfly sign bridges consisted of either a single chord, or double chord without web members. Other sign bridges generally have truss members consisting of four round chord and angle web members supporting signs on the span or arm (although some three chord structures have been used for full span sign bridges). Towers are comprised of one column for a butterfly, cantilever and full span three chord sign bridges. Full span four round chord sign bridge towers usually consist of two columns joined by angle web members at each end of the span. All “Sign Bridges” are designed by the Bureau of Structures or a consultant. Structure contract plans provide full details that a fabricator can construct the sign bridge from. Standard details for the full span four chord sign bridge associated with this Chapter of the WisDOT Bridge Manual require a design for each sign bridge structure including foundations. Standard design and details for steel cantilever sign bridge and footing are available for use without performing individual design if a structure meets the limitations required by the standards. These details are used for type I and II sign applications only.

Sign bridges carrying DMS require special consideration. Special concerns include:

1. Size and weight of the sign panel, and attachment location with respect to the axis of the truss.

2. Size and weight of catwalk, and attachment location with respect to the axis of the truss.

3. Consideration of wind effects unique to these signs.

4. Modification to support brackets. All catwalk and sign bracket connections shall be made with friction type connections and high strength A325 bolts with DTI washers.

Wisconsin recommends the use of the Minnesota four chord steel angle truss configuration for sign bridges carrying DMS, providing that the designer checks the design of each member and connection details and make necessary modification to conform to the latest AASHTO Standard specification requirements as stated in Section 39.2. Each foundation shall be designed and included in the contract plans with the sign bridge structures.

“Overhead Sign Supports” are smaller sign structures carrying type II (smaller) directional signs, limited amounts of type I signs and small LED or changeable message signs. Type II sign depths have ranged from 3'-0" to 4'-0" deep for traffic directional signs, and up to 10'-0" for small information type I signs. When a sign is larger than 10'-0" deep, the structure is to be designed as a sign bridge. Cantilever overhead sign supports accommodated up to 45 sq. ft. of sign area. Total sign areas accommodated on full span overhead sign supports range up to 300 sq. ft. These ranges are again an approximate guide and can be more or less depending on variables such as span length, location of the sign with respect to the tower(s) the height of the tower(s), etc. Towers are comprised of single column (uniform or tapered pipe) for either the cantilever or full span overhead sign support. Arms on cantilever or the span on a full span overhead sign support are either one chord (uniform or tapered pipe), or two chords with or without angle web members depending on the span length and sign depth. Due to the variability of factors that can influence the selection of structure type, designers are encouraged to contact BOS Structures Design Section for further assistance when sign areas fall outside of the above limits, or when structural geometry is in question. "Overhead Sign Supports" are normally bid by contractor and designed by a fabricator or by another party for a fabricator to construct. Typical structures with steel poles on standard concrete bases usually have the least plan detail associated with them and are normally depicted in the Construction Detail portion of the state contract plans. However, it is recommended that plan development for projects with multiple structures, such as major or mega projects, and structures mounted on non-standard supports to be prepared by structural engineers and placed in section 8 of the contract plans along with the sign bridge plans. When a standard concrete base design is required the corresponding SDD sheets shall be used as drawings, and they must be inserted into the contract plans for overhead sign supports. See the WisDOT FDM Procedures 11-55-20 and 15-1-20 for more information on "Overhead Sign Supports".

39.6 Geotechnical Guidelines

Several potential problems concerning the required subsurface exploration for foundations of sign structures exist. These include:

- The development and location of these structures are not typically known during the preliminary design stage, when the majority of subsurface exploration occurs. This creates the potential for multiple drilling mobilizations to the project.
- Sometimes these structures are located in areas of proposed fill soils. The source and characteristics of this fill soil is unknown at the time of design.
- The unknowns associated with these structures in the scoping/early design stages complicate the consultant contracting process. How much investigation should be scoped in the consultant design contract?

Currently, all sign structure foundations are completely designed and detailed in the project plans. Sign-related design information can be found in the Facilities Development Manual (FDM) or Bridge Manual as described in the following sections.

WisDOT policy item:

The length of a cast-in-place shaft foundation shall be limited to 20’-0” for both sign bridges and overhead sign supports. Deviation from this policy item may be allowed provided coordination is made with BOS Structures Design Section.

39.6.1 Sign Bridges

WisDOT has created a standard foundation design for cantilever sign bridges carrying Type I and II signs. This standard foundation is presented on Standard 39.12 of the Bridge Manual Standard Details. The wings on this single shaft footing are used to help resist torsion. If a cantilever sign bridge exceeds the criteria/limitations (shown on Standard 39.10), the standard foundation shall not be utilized, and an individual foundation must be fully designed. This customized design will involve determining the subsurface conditions as described in section 39.6.3.

Foundations supporting all butterfly and full span sign bridges are custom designed. They generally have two cylindrical shafts connected by a concrete cross-girder below the columns. Other foundations such as single shaft, pile foundation and spread footing may be detailed when subsurface condition, constructability issue or economic present a more desirable design. WisDOT has no standard details for the foundations of these structures.

39.6.2 Overhead Sign Supports

Overhead sign supports are described in Sections 11-55-20 and 15-1-20 of the FDM. In addition, Section 641 of the Standard Specifications outlines the design/construction aspects of these structures.

If these structures are carrying type I and II signs, and meeting several criteria/limitations that are listed on the SDD's, the designer can use WisDOT-developed standard foundations for them. The designer can then insert the proper SDD sheet into the plans. SDD sheets exist for cantilever overhead sign supports. These single shaft bases for cantilever overhead sign supports vary in depth and range from 24" to 42" in diameter (SDD 15c22-2 thru 15c25-2). Another SDD sheet applies to full-span overhead sign supports and is 36" in diameter (SDD 15c15-3). The standard foundations in these SDD sheets were designed using slightly conservative soil design parameters. If the design criteria for these standard designs are not met, the SDD sheets cannot be used, the structure foundation must be fully designed and the unique details shall be done in accordance to the overhead sign support mounted on non-standard supports procedure described in Section 39.5. This involves determining the subsurface conditions as described in the following section.

39.6.3 Subsurface Investigation and Information

No subsurface investigation/information is necessary for any of the sign structures that meet the limitations for allowing the use of WisDOT standard foundations. Appropriate subsurface information is necessary for any of these structures that require custom designs.

There may be several methods to obtain the necessary subsurface soil properties to allow for a custom design of foundations, as described below:

- In areas of fill soils, the borrow material may be unknown. The designer should use their best judgment as to what the imported soils will be. Standard compaction of this material can be assumed. Conservative soil design parameters are encouraged.

- The designer may have a thorough knowledge of the general soil conditions and properties at the site and can reasonably estimate soil design parameters.

- The designer may be able to use information from nearby borings. Judgment is needed to determine if the conditions present in an adjacent boring(s) are representative of those of the site in question.

- If the designer cannot reasonably characterize the subsurface conditions by the above methods, a soil boring and Geotechnical report (Site Investigation Report) should be completed. Necessary soil design information includes soil unit weights, cohesions, phi-angles and location of water table.

Designers, both internal and consultant, should also be aware of the potential of high bedrock, rock fills and the possible conflict with utilities and utility trenches.

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40.3 Bridge Replacements

Bridge rehabilitation is preferred over bridge replacement if the final structure provides adequate serviceability. In order to obtain federal funding eligibility for rehabilitation or replacement; the bridge must be Structurally Deficient or Functionally Obsolete. The Federal Sufficiency Number is a guide for federal participation which is required to be less than 50 for replacement. Also, Wisconsin DOT requires the Rate Score to be less than 75. Bridges are not eligible for replacement unless the Substructure or Superstructure Condition is 4 or less or the Inventory Rating is less than HS10 or the Alignment Appraisal is 4 or less.

A bridge becomes Structurally Deficient when the condition of the deck, superstructure or substructure is rated 4 or less; or when the inventory load capacity is less than 10 tons (89.0 kN); or when the waterway adequacy is rated a 2.

A bridge becomes Functionally Obsolete when the bridge roadway width, vertical clearance, or approach alignment is substandard (appraisal rating of 3 or less), or when the inventory load capacity is less than 15 tons; or when the waterway adequacy is rated a 3 or less.

See FDM 11-40-1, 1.5 for policies regarding necessary bridge width and structural capacity to determine eligibility for bridge rehabilitation* versus bridge replacement.

* If lane widening is planned as part of the 3R project, the usable bridge width should be compared to the planned width of the approaches after they are widened.

40.4 Rehabilitation Considerations

As a structure ages, rehabilitation is a necessary part of insuring some level of acceptable serviceability. The first consideration for any bridge rehabilitation decision is whether its geometry and load carrying capacity are sufficient to safely carry present and projected traffic. Information necessary to determine structure sufficiency includes structure inspection, inventory, traffic, maintenance, capacity and functional adequacy. The methods of reconstruction are based on the type of structures, existing condition or rating information, the preliminary details of rehabilitation, and traffic control costs. These are important factors in considering rehabilitation options such as either a deck protection system or a deck replacement.

The Regions are to evaluate bridge deficiencies when the bridge is placed in the program to insure that rehabilitation will remove all structural deficiencies. FHWA requires this review and Bureau of Structures (BOS) concurrence with all proposed bridge rehabilitation. On high cost bridges, a closer check of the Functionally Obsolete Criteria may be required. On high cost bridges a 2' shoulder is acceptable on a low speed, low volume roadway having a good accident record. After rehabilitation work is completed, the bridge should not be Structurally Deficient or Functionally Obsolete. A sufficiency number greater than 80 is also required after completion of the rehabilitation work. However, if conditions exist that would prevent the completed improvement from correcting all deficiencies, WisDOT shall determine if the proposed project is eligible based on safety and the public interest. Contact the Bureau of Structures Development Section for a waiver of the sufficiency number requirement.

<u>WisDOT policy item:</u>

Rate the bridge using LFR provided it was designed using ASD or LFD. There are instances, however, where the LRFR rating of an existing bridge is beneficial (e.g. There is no M/M_u reduction to shear capacity using LRFR, which can affect longer-span steel bridges). Please contact the Bureau of Structures Development Section if using LRFR to rate bridges designed using ASD or LFD. Bridges designed using LRFD must be rated using LRFR.

Deck removal on prestressed girders is a concern as the contractors tend to use jackhammers to remove the deck. Contractors have damaged the top girder flanges using this process either by using too large a hammer or carelessness. With the wide-flange sections this concern is amplified. It is therefore suggested that the contractor saw cut the slab to be removed longitudinally close to the shear connectors. With the previously applied bond breaker, the slab should break free and then the contractor can clear the concrete around the shear connectors. Saw cutting needs to be closely monitored as contractors have sawn through steel girder flanges by not watching their blade depth.

In the rehabilitation or widening of bridge decks, it is often necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibrations on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. Wisconsin bridge construction experience indicates that there are many cases where problems have occurred during deck pours with adjacent traffic. Consideration should be given to prohibiting traffic during the deck pour until concrete has taken its initial set.

40.6 Deck Replacements

Depending on the structure age or site conditions, the condition of original deck or deck overlay, a complete deck replacement may be the most cost effective solution and extend the life of the bridge by 40 years or more. Epoxy coated rebars are required on bridge deck replacements under the same criteria as for new bridges. The new deck and parapet or railing shall be designed per the most recent edition of the WisDOT Bridge Manual, including continuity bars and overhang steel. Refer to the criteria in Section 40.3 of this chapter for additional 3R project considerations. The top flange of steel girders should be painted.

The following condition or rating criteria are the minimum requirements on STN bridges eligible for deck replacements:

Item	Existing Condition	Condition after Construction
Deck Condition	≤ 4	≥ 8
Inventory Rating	---	≥ HS15*
Superstructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Substructure Condition	≥ 3	Remove deficiencies (≥ 8 desired)
Horizontal and Vertical Alignment Condition	> 3	---
Shoulder Width	6 ft	6 ft

Table 40.6-1
Condition Requirements for Deck Replacements

*Rating evaluation is based on the criteria found in Chapter 45-Bridge Rating. An exception to requiring a minimum Inventory Rating of HS15 is made for continuous steel girder bridges, with the requirement for such bridges being a minimum Inventory Rating of HS10. For all steel girder bridges, assessment of fatigue issues as well as paint condition (and lead paint concerns) should be included in the decision as to whether a deck replacement or a superstructure/bridge replacement is the best option.

For any bridge not meeting the conditions for deck replacement, a superstructure, or likely a complete bridge replacement is recommended. For all Interstate Highway deck replacements, the bridges are to have a minimum Inventory Rating of HS20 after the deck is replaced.

WisDOT policy item:

Contact the Bureau of Structures Development Section Ratings Unit if a deck replacement for an Interstate Highway bridge would result in an Inventory Rating greater than HS18, but less than HS20.

For slab superstructure replacements on concrete pier columns, it is reported that the columns often get displaced and/or cracked during removal of the existing slab. Plans should be detailed showing full removal of these pier columns to the existing footing and replacement with current standard details showing a concrete cap with pier columns or shaft.

See the *Facilities Development Manual* and *FDM SDD 14b7* for anchorage/offset requirements for temporary barrier used in staged construction. Where temporary bridge barriers are being used, the designer should attempt to meet the required offsets so that the barrier does not require anchorage which would necessitate drilling holes in the new deck.

In general, the substructure need not be analyzed for additional dead load provided the new deck is comparable to the existing. Exceptions include additional sidewalk or raised medians that would significantly alter the dead load of the superstructure.

For prestressed girder deck replacements, replace intermediate concrete diaphragms with steel diaphragms in locations of removed diaphragms (i.e. don't add intermediate lines of diaphragms). See Chapter 19 Standard Details and Steel Diaphragm Insert Sheets for additional information.

For deck replacement projects that change global continuity of the structure, the existing superstructure and substructure elements shall be evaluated using LFD criteria. One example of this condition is an existing, multi-simple span structure with expansion joints located at the pier locations. From a maintenance perspective, it is advantageous to remove the joints from the bridge deck and in order to do so; the continuity of the deck must be made continuous over the piers.

355.4. For adhesive anchors located in a region of a concrete member where analysis indicates no cracking at service load levels, τ_{uncr} shall be permitted to be used in place of τ_{cr} and shall be taken as the 5 percent fractile of results of tests performed and evaluated according to ICC-ES AC308 / ACI 355.4.

In addition to the checks listed above for all adhesive anchors, for “Type L” anchors, the factored sustained tensile force must be less than or equal to the factored sustained tensile resistance per **ACI [D.4.1.2]**:

$$0.55\phi_{tc}N_{ba} \geq N_{ua,s}$$

40.16.5 Concrete Masonry Anchor Shear Capacity

Concrete masonry anchors in shear fail in one of three ways: steel shear rupture, concrete breakout, or concrete pryout. Figure 40.16-3 shows the concrete breakout failure mechanism for anchors in shear.

The projected concrete breakout area, A_{Vc}, shown in Figure 40.16-3 is limited vertically by H, and in both horizontal directions by S_i:

H = Minimum of:

1. The member depth (h_a) or
2. 1.5 times the edge distance (c_{a1}) (in).

S_i = Minimum of:

1. Half the anchor spacing (S),
2. The perpendicular edge distance (c_{a2}), or
3. 1.5 times the edge distance (c_{a1}) (in).

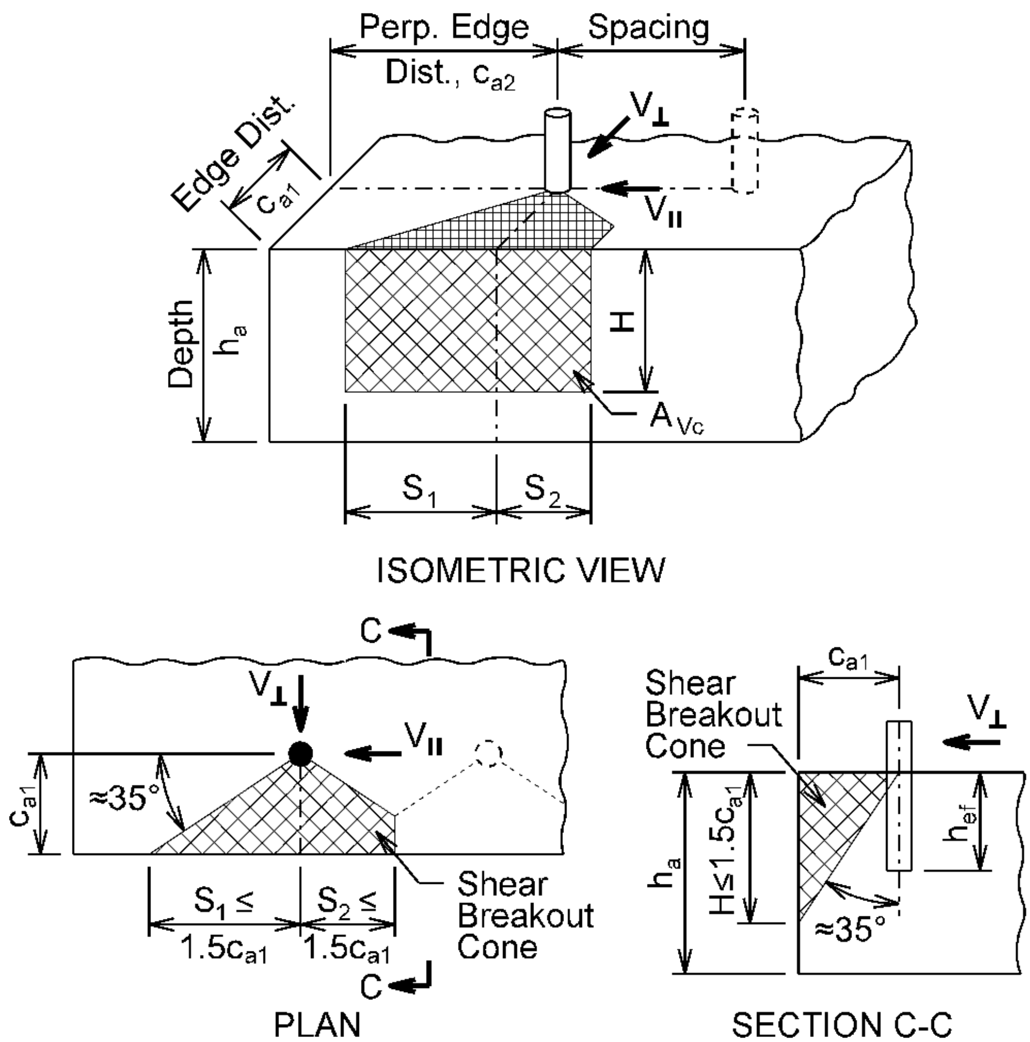

Figure 40.16-3
Concrete Breakout of Concrete Masonry Anchors in Shear

If the shear is applied to more than one row of anchors as shown in Figure 40.16-4, the shear capacity must be checked for the worst of the three cases. If the row spacing, SP, is at least equal to the distance from the concrete edge to the front anchor, E1, check both Case 1 and Case 2. In Case 1, the front anchor is checked with the shear load evenly distributed between the rows of anchors. In Case 2, the back anchor is checked for the full shear load. If the row spacing, SP, is less than the distance from the concrete edge to the front anchor, E1, then check Case 3. In case 3, the front anchor is checked for the full shear load. If the anchors are welded to an attachment to evenly distribute the force to all anchors, only Case 2 needs to be checked.

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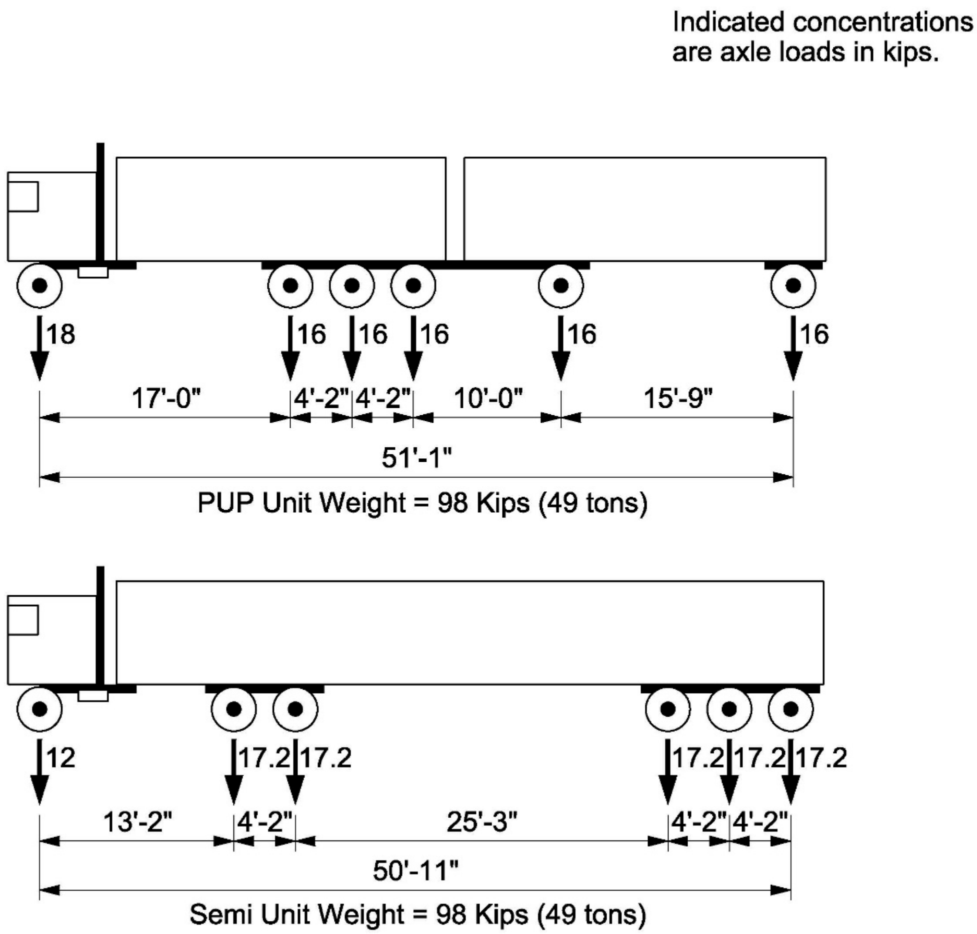

Figure 45.4-3
WisDOT Specialized Annual Permit Vehicles

45.4.1 Posting Live Loads

The live load to be used in the rating formula for posting considerations should be any of the three typical AASHTO Commercial Vehicles (Type 3, Type 3S2, Type 3-3) shown in Figure 45.4-1, any of the four AASHTO Specialized Hauling Vehicles (SU4, SU5, SU6, SU7) shown in Figure 45.4-2, the Wisconsin Standard Permit Vehicle shown in Figure 45.6-1, or in certain cases the specialized annual permit vehicles shown in Figure 45.4-3.

As stated in **MBE [6A.4.4.2.1a]**, for spans up to 200', only the vehicle shall be considered present in the lane for positive moments. It is unnecessary to place more than one vehicle in a lane for spans up to 200' because the load factors provided have been modeled for this possibility. For spans 200' in length or greater, the AASHTO Type 3-3 truck multiplied by 0.75 shall be analyzed combined with a lane load as shown in Figure 45.4-4. The lane load shall be taken as 0.2 klf in each lane and shall only be applied to those portions of the span(s) where the loading effects add to the vehicle load effects.

Also, for negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 trucks multiplied by 0.75 shall be used. The trucks should be heading in the same direction and should be separated by 30 ft as shown in Figure 45.4-4. There are no span length limitations for this negative moment requirement.

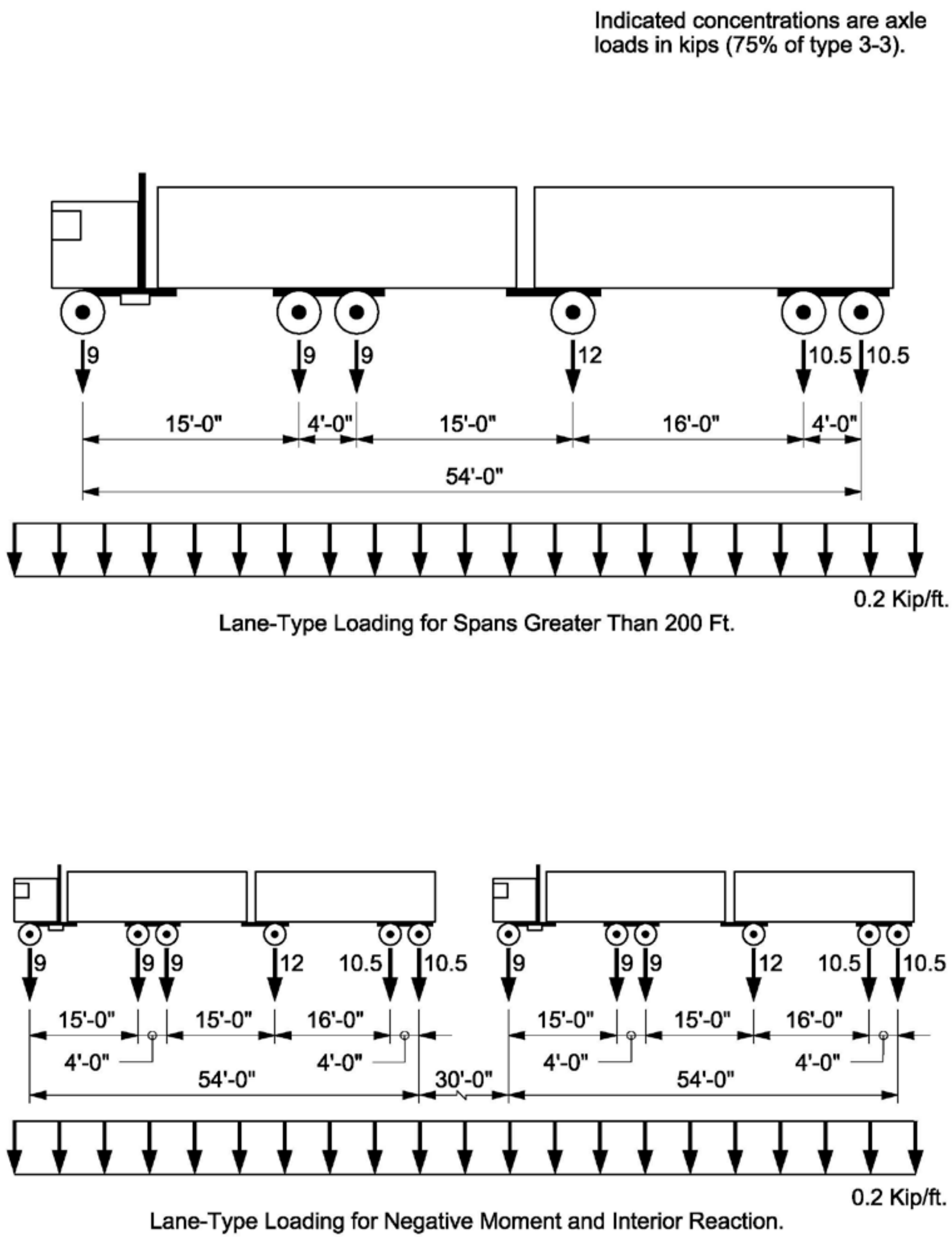

Figure 45.4-4
Lane Type Legal Load Models

45.4.2 Posting Signage

Current WisDOT policy is to post State bridges for only one tonnage capacity. Bridges which cannot carry the maximum weight for the vehicles described in Section 45.4.1 using Operating Rating criteria are posted with one of the standard signs, shown in Figure 45.4-5 showing the bridge capacity for the governing vehicle, which should conform to the requirements of the *Wisconsin Manual for Uniform Traffic Control Devices (WMUTCD)*.

In the past, local bridges were occasionally posted with the signs shown in Figure 45.4-6 using the H20, Type 3 and Type 3S2 vehicles. The H20 represented the two-axle vehicle, the Type 3 represented the three-axle vehicle and the Type 3S2 represented the combination vehicle. This practice is not encouraged by WisDOT and is generally not allowed for State owned structures, except with permission from the State Bridge Engineer.

WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

Figure 45.4-5
Standard Signs Used for Posting Bridges

WEIGHT LIMIT 2 AXLE VEHICLES 15 TONS 3 AXLE VEHICLES 20 TONS COMBINATION VEHICLES 30 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS	WEIGHT LIMIT 2 AXLE VEHICLES 14 TONS 3 AXLE VEHICLES 18 TONS COMBINATION VEHICLES 28 TONS

Figure 45.4-6
Historic Load Posting Signs

45.5 Material Strengths and Properties

Material properties shall be as stated in AASHTO *MBE* or as stated in this chapter. Often when rating a structure without a complete set of plans, material properties are unknown. The following section can be used as a guideline for the rating engineer when dealing with structures with unknown material properties. If necessary, material testing may be needed to analyze a structure.

45.5.1 Reinforcing Steel

The allowable unit stresses and yield strengths for reinforcing steel can be found in Table 45.5-1. When the condition of the steel is unknown, they may be used without reduction. Note that Wisconsin started to use Grade 40 bar steel about 1955-1958; this should be noted on the plans.

Reinforcing Steel Grade	Inventory Allowable (psi)	Operating Allowable (psi)	Minimum Yield Point (psi)
Unknown	18,000	25,000	33,000
Structural Grade	19,800	27,000	36,000
Grade 40 (Intermediate)	20,000	28,000	40,000
Grade 60	24,000	36,000	60,000

Table 45.5-1
Yield Strength of Reinforcing Steel

45.5.2 Concrete

The following are the maximum allowable unit stresses in concrete in pounds per square inch (see Table 45.5-2). Note that the “Year Built” column may be used if concrete strength is not available from the structure plans.

45.7.3 Single Trip Permit Information

Non-divisible loads which exceed the annual permit restrictions may be moved by the issuance of a single trip permit.

When a single trip permit is issued, the applicant is required to indicate on the permit the origin and destination of the trip and the highways that are to be used. Another permit is needed for local roads. Each Single Trip Permit vehicle is individually analyzed by WisDOT for all structures that it encounters on the designated permit route.

The load distribution is based on single lane distribution. This is used because these permit loads are infrequent and are likely the only heavy loads on the structure during the crossing. The analysis is done at the operating level.

In special cases the dynamic load allowance (or impact for LFR) may be neglected provided that the maximum vehicle speed can be reduced to 5 MPH prior to crossing the bridge. Also, if some cases, the truck may be escorted across the bridge with no other vehicles allowed on the bridge during the crossing. If this is the case, then the live load factor can be reduced from 1.20 to 1.10 as shown in Table 45.3-3. It is recommended that the truck be centered on the bridge if it is being escorted with no other vehicles allowed on the bridge during the crossing.

45.8 Load Rating Documentation

45.8.1 Load Rating Summary Sheet

After the structure has been load rated, the WisDOT Bridge Load Rating Summary Form shall be completed and utilized as a cover sheet for the load rating calculations (see Figure 45.8-1). This form may be obtained from the Bureau of Structures or is available on the following website:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/plan-submittal.aspx

Note: The Load Rating Summary Form is not required to be completed and sent in for concrete box culvert structures.

Instructions for completing the form are as follows:

1. Check what method was used to rate the bridge in the space provided.

2. Enter all data for all items corresponding to the vehicle type. Capacities for the posting vehicles do not have to be calculated if the Operating Rating factor is greater than 1.0 for the HL-93 (LRFR) or the HS20 (LFR or ASR).

3. The rating for the Wis-SPV is always required and should be given on the rating sheet for both a multi-lane distribution and a single-lane distribution. Make sure not to include the future wearing surface in these calculations. All reported ratings are based on current conditions and do not reflect future wearing surfaces.

4. For the Operating Rating, enter the lowest rating for each appropriate vehicle type, subject to the above requirements.

5. For the controlling element, make sure to enter the element (slab, deck girder, lower truss chord, etc.) as well as the check (moment, shear, etc).

6. Be specific in describing where the controlling rating is located. For example, for girder bridges, enter the controlling span, girder-line, and location within the span (Ex. Span 2, G3, midspan).

7. For the live load distribution factor, enter the distribution factor for the controlling element. Be sure to specify if it is a shear distribution factor or a moment distribution factor.

8. Enter all additional remarks as required to clarify the load capacity calculations and, if necessary, recommend posting signage.

9. It is necessary for the responsible engineer to sign and seal the form in the space provided for projects where the Ratings have changed. However, for rehabilitation projects with no change to the Ratings, the Load Rating Summary Form does not need to be signed and sealed.

The factored shear force at the critical section is: $\boxed{V_{u\_crit} = 352}$ kips

E45-2.9 Design Load Rating

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Live Load Factors taken from Table 45.3-1

$\gamma_{L\_inv} := 1.75$ $\gamma_{DC} := 1.25$ $\gamma_{servLL} := 0.8$

$\gamma_{L\_op} := 1.35$ $\phi_c := 1.0$ $\phi_s := 1.0$

$\phi := 1.0$ for flexure

$\phi := 0.9$ for shear

For Flexure

Inventory Level

$$RF_{Mom\_Inv} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_inv} \cdot (M_{LLIM})}$$

$\boxed{RF_{Mom\_Inv} = 1.723}$

Operating Level

$$RF_{Mom\_Op} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_op} \cdot (M_{LLIM})}$$

$\boxed{RF_{Mom\_Op} = 2.233}$

For Shear at first critical section

Inventory Level

$$RF_{shear\_Inv} := \frac{(1)(1)(0.9)(V_n) - \gamma_{DC} \cdot (V_{DCnc} + V_{DCc})}{\gamma_{L\_inv} \cdot (Vi_{LL})}$$

$\boxed{RF_{shear\_Inv} = 1.110}$

Operating Level

$$RF_{shear\_Op} := \frac{(1)(1)(0.9)\left(V_n\right) - \gamma_{DC}\cdot\left(V_{DCnc} + V_{DCc}\right)}{\gamma_{L\_op}\cdot\left(Vi_{LL}\right)}$$

$\boxed{RF_{shear\_Op} = 1.439}$

At the Service III Limit State (Inventory Level):

$$RF \quad = \quad \frac{f_R - \gamma_D\cdot\left(f_D\right)}{\gamma_{servLL}\left(f_{LLIM}\right)}$$

$T := ns\cdot A_s\cdot f_{pe}$ $\boxed{T = 1626}$ kips

$f_{pb} := \frac{T}{A_g} + \frac{T\cdot\left(e_s\right)}{S_b}$ $\boxed{f_{pb} = 4.414}$ ksi

Allowable Tensile Stress

$t_{all} := -0.19\cdot\sqrt{f'_c}$; $|\ t_{all}\ | \le 0.6$ ksi $\boxed{t_{all} = -0.537}$ ksi

$f_R := f_{pb} - t_{all}$ $\boxed{f_R = 4.951}$ ksi

Live Load Stresses:

$f_{LLIM} := \frac{M_{LLIM}\cdot 12}{S_{cgb}}$ $\boxed{f_{LLIM} = 1.496}$ ksi

Dead Load Stresses:

$f_{DL} := \frac{M_{DC1}\cdot 12}{S_b} + \frac{M_{DC2}\cdot 12}{S_{cgb}}$ $\boxed{f_{DL} = 3.240}$ ksi

$RF_{serviceIII} := \frac{f_R - 1.0\cdot\left(f_{DL}\right)}{\gamma_{servLL}\cdot\left(f_{LLIM}\right)}$ $\boxed{RF_{serviceIII} = 1.430}$

E45-2.10 Legal Load Rating

Since the Operating Design Load Rating RF>1.0, the Legal Load Rating is not required. The Legal Load computations that follow have been done for illustrative purposes only. Shear ratings have not been illustrated.

Live Loads used will be the AASHTO Legal Loads per Figure 45.4-1 and AASHTO Specialized Hauling Vehicles per Figure 45.4-2.

$\boxed{g_i = 0.636}$

$IM := 33$ %

* WisDOT does not allow for a dynamic load allowance reduction based on the smoothness of the roadway surface. Thus, IM=33%

At the Strength I Limit State:

$$RF = \frac{(\phi_c)(\phi_s)(\phi)R_n - \gamma_{DC}(DC_1) - \gamma_{DW}(DW_1)}{\gamma_L(LL + IM)}$$

Live Load Factors taken from Tables 45.3-1 and 45.3-2

$\phi_c := 1.0$ $\phi_s := 1.0$

$\phi := 1.0$

$\gamma_{L\_Legal} := 1.45$ $\gamma_{DC} := 1.25$

$\gamma_{L\_SU} := 1.45$

For Flexure

$$RF_{Legal} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_Legal} \cdot (M_{LLIM})}$$

$$RF_{SU} := \frac{(1)(1)(1)(M_n) - \gamma_{DC} \cdot (M_{DC1} + M_{DC2})}{\gamma_{L\_SU} \cdot (M_{LLIM})}$$

AASHTO Type	Truck Type	Truck Weight (Tons)	M_{LL} (Per Lane) (ft-kips)	M_{LLIM} (M_{LL} * IM * g_i) ft-kips	RF Strength I Flexure	Safe Load Capacity (Tons)	Posting?
Commercial Trucks	Type 3	25	1671.0	1413.4	4.520	113	No
	Type 3S2	36	2150.0	1818.6	3.513	126	No
	Type 3-3	40	2260.0	1911.7	3.342	134	No
Specialized Hauling Vehicles	SU4	27	1831.0	1548.8	4.124	111	No
	SU5	31	2062.8	1744.9	3.661	113	No
	SU6	34.75	2294.6	1940.9	3.291	114	No
	SU7	38.75	2540.8	2149.2	2.972	115	No

As expected, all rating factors are well above 1.0. However, if any of the rating factors would have fallen below 1.0, the posting capacity would have been calculated per 45.3.2.7.2:

$$\text{Posting} := \left(\frac{W}{0.7}\right)[(RF) - 0.3]$$

E45-2.11 Permit Load Rating

For any bridge design (new or rehabilitation) or bridge re-rate, the Wisconsin Standard Permit Vehicle (Wis-SPV) shall be analyzed per 45.6.

The bridge shall be analyzed for this vehicle considering both single-lane and multi-lane distribution. Also, the vehicle will be analyzed assuming it is mixing with other traffic on the bridge and that full dynamic allowance is utilized. Future wearing surface shall be included.

Since this example is rating a newly designed bridge, an additional check is required. The designer shall ensure that the results of the single-lane analysis are greater than 190 kips MVW.

Also, divide out the 1.2 multiple presence factor per **LRFR [6.4.5.4.2.2]** for the single lane distribution factor run.

For 146' span:

$M190_{LL} := 4930.88$ kip-ft per lane

$V190_{LL} := 145.08$ kips at $\boxed{d_v = 65}$ in

for Strength Limit State

Single Lane Distribution w/ Future Wearing surface (Design check per 45.6)

$g_{m1} := 0.435\frac{1}{1.2}$ $\boxed{g_{m1} = 0.363}$

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E45-4 Steel Girder Rating Example - LRFR

This example shows rating calculations conforming to the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* as supplemented by the *WisDOT Bridge Manual (July 2008)*. This example will rate the design example E24-1 contained in the WisDOT Bridge Manual. **(Note: Example has not been updated for example E24-1 January 2016 updates)**

E45-4.1 Preliminary Data

An interior plate girder will be rated for this example. The girder was designed to be composite throughout. There is no overburden on the structure. In addition, inspection reports reveal no loss of section to any of the main load carrying members.

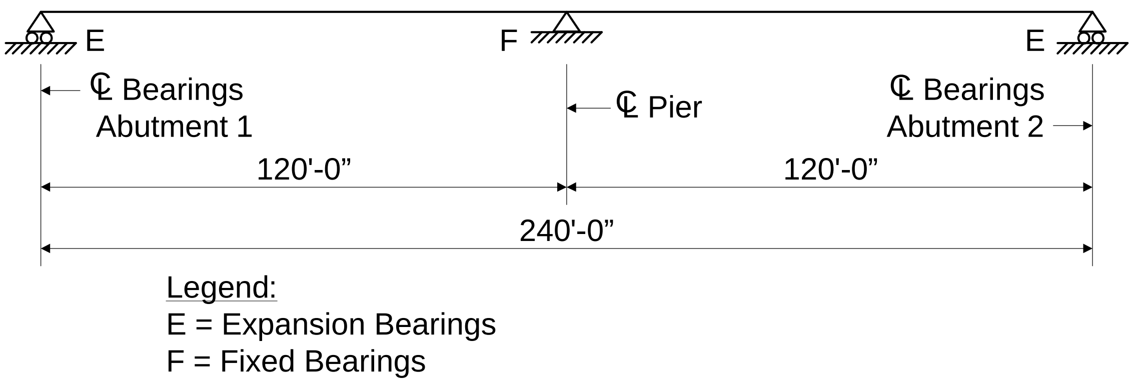

Figure E45-4.1-1
Span Configuration

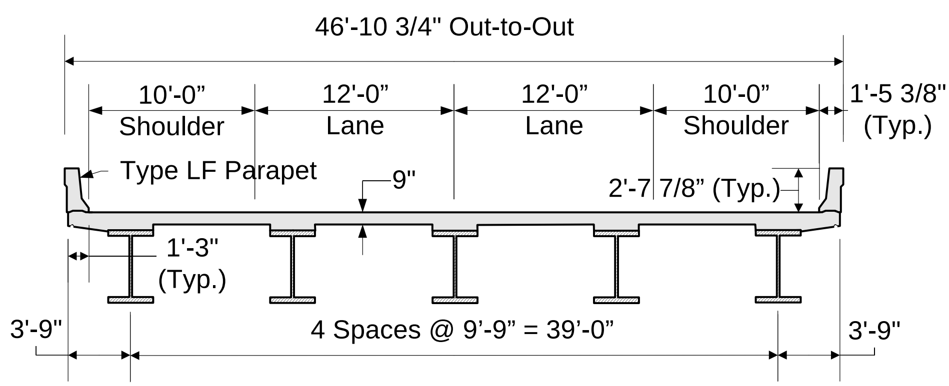

Figure E45-4.1-2
Superstructure Cross Section